THE MECHANICAL TREATMENT OF TOPSOILS

FOR USE IN HIGHWAY EMBANKMENTS

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UNIVERSITY OF ALBERTA

FACULTY OF APPLIED SCIENCE

This is to certify that the undersigned have read and recommended to the Committee on Graduate Studies for acceptance, a thesis submitted by R. J. Hollingshead, B.Sc. "1946", entitled:

The Mechanical Treatment of Top Soils for Use in Highway Embankments.

T H E S I S

THE MECHANICAL TREATMENT OF TOP SOILS FOR USE IN HIGHWAY EMBANKMENTS

Submitted as the Partial Fulfillment

of the

Requirements for the Degree of

Master of Science

by

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Under the direction of

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Edmonton, Alberta.

April 10th, 1948.

R. J. Hollingshead

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FOREWARD

The rapid extension of the systems of hard-surfaced roads has provided the incentive for thorough subgrade investigations prior to the design of the roadway.

In contrast to foundation subsoils, highway subgrades are considerably influenced by climatic conditions. Because of this, general conclusions based on the results of laboratory tests can be very misleading.

From the ground surface to a depth of approximately 6 feet, the physical properties of the soil are influenced by seasonal changes of temperature and moisture and by agents such as roots, worms, and bacteria. The upper part of this region is known as the A-horizon. It is subjected primarily to the effects of weathering. The lower part is referred to as the B-horizon. The highway engineer is chiefly concerned with the soils in these two horizons.

Numerous authorities in Canada and the U. S. A. have recommended the wasting of topsoils in highway and earth dam work. This recommendation involves stripping and hauling of this topsoil to some place where it may be wasted, which procedure can be very costly.

During the past two summers the writer has been employed by the Department of Public Works, Province of Alberta. In that time, he has observed and experienced considerable construction difficulty in base course stabilization at some of the localities where topsoils were present in the embankment. Where the topsoil has been wasted in highway construction over virgin territory, the cost of grading per mile has been increased by as much as three times. Difficulty is also experienced in cases where the grade is being rebuilt. Usually drifting soil from cultivated fields has filled the ditches of the old grade. The amount of topsoil to be handled is thus greatly increased.

In preparing a program of soil study the magnitude of the job must be considered. If the proposed construction involves only a small expenditure, the designer cannot afford more than a few classification tests on representative samples. This lack of information must be compensated for by a liberal factor of safety in design. However, where the expenditure is large the cost of a thorough investigation is usually small compared to the savings possible by utilizing the results of the study in design and construction. Hence, on large projects, extensive soil investigations are quite likely justified.

Accordingly, at the suggestion of officials of the Department of Public Works of the Province of Alberta, the writer undertook to conduct a laboratory study, under the direction of Dean R. M. Hardy, of compacted topsoil and subsoil samples, obtained from various localities in the Province of Alberta. This study was undertaken for the purpose of recommending some safe and practical solution to this difficult problem. If such a solution was not apparent at the completion of the study, it was hoped that the information obtained would suggest a program of future research.

ACKNOWLEDGEMENTS

Arrangements were made with Mr. L. H. McManus, Testing Engineer, Department of Public Works, Province of Alberta, to procure the necessary soil samples from the various parts of the province.

The laboratory study and analysis of results were accomplished under the direction of Dean R. M. Hardy, University of Alberta.

The writer desires to express his appreciation for the assistance and cooperation received from Mr. McManus and Dean Hardy.

Special mention should be made of the cooperation received in the laboratory from Mr. S. R. Sinclair, of the Department of Civil Engineering, at the University of Alberta.

INTRODUCTION

In general, soil testing can be broken down into two general classes:

1. Classification or identification tests, which give results that can be used in semi-empirical design. They may also assist in reducing the number of more elaborate tests that would be necessary to predict the behaviour of the soil.

2. Tests that give numerical values for a soil property. Such results can be used in theoretical soil mechanics as stresses are used in steel design. Examples of these soil properties are load bearing capacity, effect of changes in moisture, rate of flow of water through the soil, or its permeability, effect of freezing conditions, stress and deformation relations.

Only classification tests for soils have been standardized by authorities such as the American Society for Testing Materials.

Atterberg Limit and Specific Gravity tests were performed on the soils received. Upon considering these results, it was apparent that the topsoil and subsoil from each location, excepting location B, were essentially the same. Since the topsoil from location B is highly compressible with a high organic content, and the subsoil from location B is a clay with medium compressibility, it was decided to concentrate on these two soils in the laboratory study.

The classification tests indicate that the topsoil and subsoil of location A are very similar. It was decided then to investigate the effect of any organic material present in the topsoil on the density and strength results.

Tabular summaries and graphical presentations of the results are shown in Section III followed by a discussion of these results in Section IV. Section V presents the conclusions of this study, and Section VI recommends a program of further study.

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I

GENERAL

LOCATION AND DESCRIPTION OF SOIL SAMPLES

Disturbed samples were received from the following locations, adjacent to existing highways:

A -- Highway 16-C-2, S.W.$\frac{1}{4}$-S4-T53-R1-W5.

B -- Highway 2-E-1, N.E.$\frac{1}{4}$-S16-T56-R25-W4, Station 1030 + 00.

C -- Highway 16-B-2, S.E.$\frac{1}{4}$-S15-T53-R23-W4, Station 2194 + 75.

D -- Station 1992 + 00, S.E.$\frac{1}{4}$-S17-T6-R24-W4.

E -- Station 2395 + 00, N.W.$\frac{1}{4}$-S22-T7-R21-W4.

F -- Station 1188 + 00, N.W.$\frac{1}{4}$-S33-T4-R23-W4.

These locations are plotted on a map of the soil zones of Alberta, Figure 1.

The soils received are briefly described as follows:

A -- Topsoil: depth 0-6 inches, a grey silty sand with a low organic matter content.

Subsoil: depth 6-24 inches, a brown silty sand with no organic matter present.

B -- Topsoil: depth 0-8 inches, a black clay with a very high organic matter content.

Subsoil: depth 8-24 inches, a grey clay with no organic matter present.

C -- Topsoil: depth 0-6 inches, a black clay with a fairly high organic matter content.

Loam Soil: depth 6-12 inches, a brown clay with no appreciable organic matter content.

Subsoil: depth 12-42 inches, a grey clay with no organic matter present.

D -- Topsoil: depth 0-6 inches, cultivated field, a black clay with appreciable organic matter content and straw.

Subsoil: depth at 24 inches, a brown silt clay with no organic matter present.

E -- Topsoil: depth 0-6 inches, a dark brown clay with straw and some organic matter present.

Subsoil: depth at 14 inches, a light brown clay with no organic matter present.

F -- Topsoil: depth 0-6 inches, a black clay with an appreciable organic matter content and straw.

Subsoil: depth at 14 inches, a brown clay with no organic matter present.

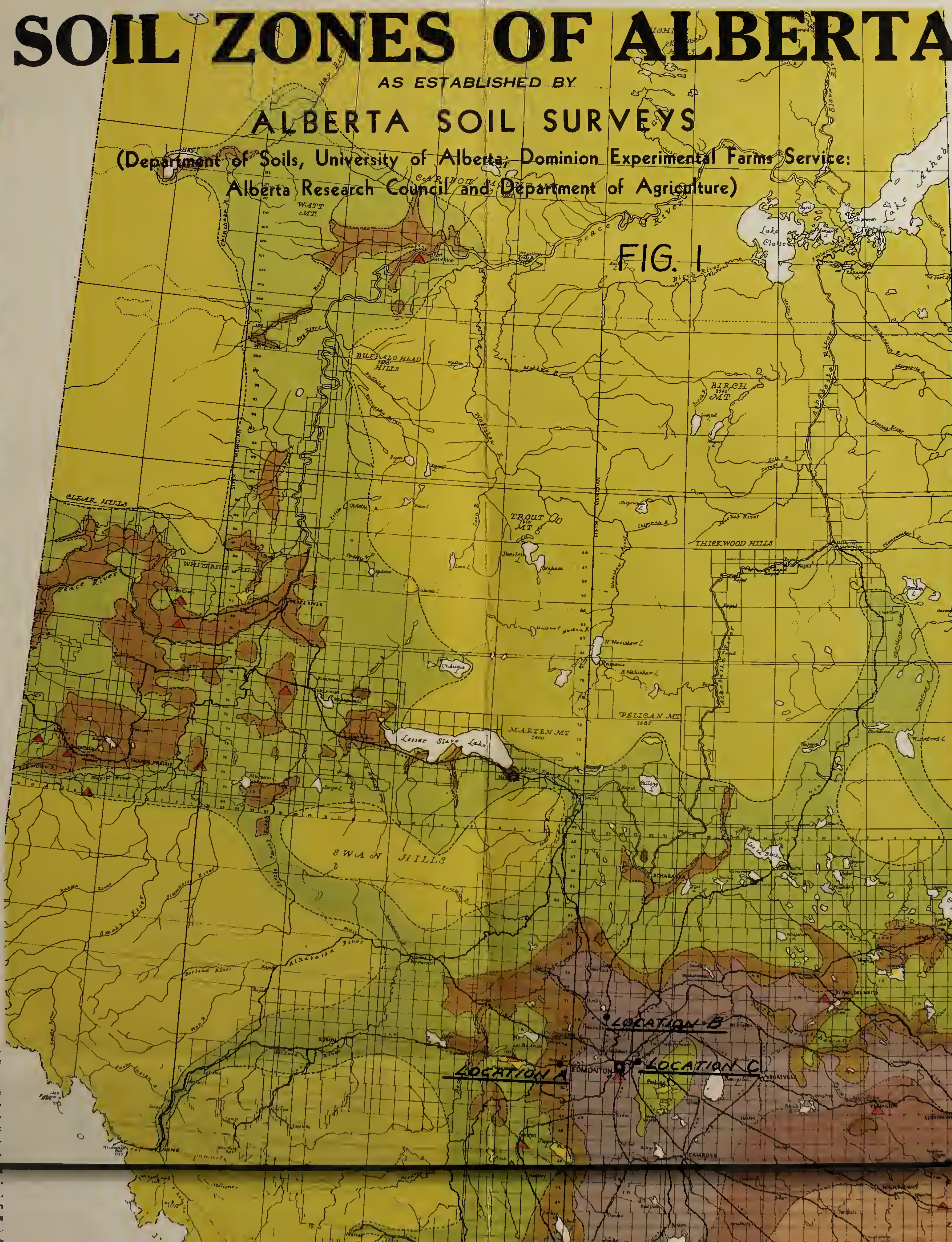
It is of interest to note that the primary highways of Alberta are built chiefly through the soil zones with the higher organic matter content, these two soil zones being the shallow black and black zones. Figure 1 shows that locations B and C are located in the black soil zone and locations D and F in the shallow black soil zone. The topsoils from these four locations possess an appreciable amount of organic matter. It would seem, therefore, that a study of a map of the soil zones does indicate just where large amounts of organic matter in the topsoil might be expected along a proposed right-of-way.

SOIL ZONES OF ALBERTA

AS ESTABLISHED BY

ALBERTA SOIL SURVEYS

(Department of Soils, University of Alberta; Dominion Experimental Farms Service: Alberta Research Council and Department of Agriculture)



BROWN

CLIMATE—Semi-arid, characterized by an average annual precipitation of 11 to 13 inches, frequent drought, high evaporation and frequent hot dry winds.

VEGETATION—Short grass prairie.

SOIL PROFILE—In the normal profile the surface (A) horizon is about 5 inches deep and brown in color. The B horizon is commonly brownish in color and lime (Bca) is found at depths averaging 15 inches below the surface. The parent material (C) is found at depths of 20 to 24 inches. In the other zones this horizon occurs at greater depths.

FERTILITY—Moisture is the principal limiting factor in crop production. Soils in this zone are relatively low in nitrogen and under irrigation often respond to phosphorus fertilizers.

LAND USE—Only the most favorable soil types can be considered arable. Most of the area is desirable for ranching. Where farmed, wheat is the principal crop grown. Cropping practices must provide for moisture conservation and control of soil drifting. The long frost-free period makes this zone a desirable area for the development of irrigation.

DARK BROWN

CLIMATE—The average annual precipitation is 13 to 15 inches, and there are less frequent droughts than in the brown zone. Fairly high evaporation and hot dry winds are added characteristics.

VEGETATION — Chiefly short grass prairie. The grass makes a denser cover and taller growth than in the brown zone.

SOIL PROFILE—In the normal profile, the surface (A) horizon averages about 7 inches in depth and is dark brown in color. The B horizon is brownish, and the lime layer (Bca) is found usually at depths of 20 to 24 inches below the surface. In this zone as in other zones, the B horizon, having received some finer materials from the A, is usually somewhat heavier and more compact than the A horizon.

FERTILITY—Moisture continues to be the principal limiting factor in crop production. Soils in this zone are relatively low in nitrogen and organic matter, but are higher in these constituents than soils of the brown zone.

LAND USE—Only the better soil types can be considered arable. The remainder generally is good pasture land. Wheat is grown almost to the exclusion of all other crops. Cropping practices must provide for conservation of moisture and control of soil drifting. The

BLACK

(Boundaries Tentative)

CLIMATE—Annual precipitation averages between 17 and 19 inches and droughts are rare. Evaporation is lower and hot winds less frequent than in the previous zones.

VEGETATION — Grassland which has been partially invaded by woodlands (mainly deciduous trees), often referred to as a parkland.

SOIL PROFILE—The normal profile has a black to very dark brown surface (A) horizon that averages about 12 to 14 inches in depth. The more compact B horizon is brown to dark brown, and the lime layer (Bca) is usually found at 30 to 40 inches below the surface.

FERTILITY—Soils in this zone are the most fertile in the province and they have in their surface foot about 3 to 4 times as much nitrogen and organic matter as there is in the average brown or gray wooded soil. Every precaution should be taken to see that they are not allowed to deteriorate.

LAND USE—A high percentage of the zone is arable. Wheat of fairly good quality can be grown, but mixed farming, including the use of fertilizer when needed, is desirable from the standpoint of both profit and permanence.

TRANSITION

(Boundaries Tentative)

CLIMATE—Annual precipitation averages from about 12 inches in the northern section to about 20 inches in the southern. Evaporation is lower than in the previous zones.

VEGETATION—Mainly woodland in which the tree growth is frequently denser and has more evergreens than in the black zone.

SOIL PROFILE—Generally quite mixed, ranging from nearly black to gray. The surface horizon consists of a thin layer of semi-decomposed litter (A0) which may be absent in burned over areas, underlain by a mineral horizon that can usually be divided into two parts. The upper (A1) part may be black, gray black or dark brown. The lower part (A2) is frequently somewhat leached of organic matter and considerably grayer than the A1. The total depth of these surface horizons averages about 10 to 12 inches. The B horizons are generally dark brown in color and lime is found at depths of about 30 to 40 inches.

as rich as those of the black zone. Leaching of the surface horizons has resulted in the loss of some plant foods.

LAND USE—A system of mixed farming that includes legumes in the crop

cover and taller growth than in the brown zone.

SOIL PROFILE—In the normal profile, the surface (A) horizon averages about 7 inches in depth and is dark brown in color. The B horizon is brownish, and the lime layer (Bca) is found usually at depths of 20 to 24 inches below the surface. In this zone as in other zones, the B horizon, having received some finer materials from the A, is usually somewhat heavier and more compact than the A horizon.

FERTILITY—Moisture continues to be the principal limiting factor in crop production. Soils in this zone are relatively low in nitrogen and organic matter, but are higher in these constituents than soils of the brown zone.

LAND USE—Only the better soil types can be considered arable. The remainder generally is good pasture land. Wheat is grown almost to the exclusion of all other crops. Cropping practices must provide for conservation of moisture and control of soil drifting. The best quality wheat in the province is grown in this and the other grassland zones.

SHALLOW BLACK

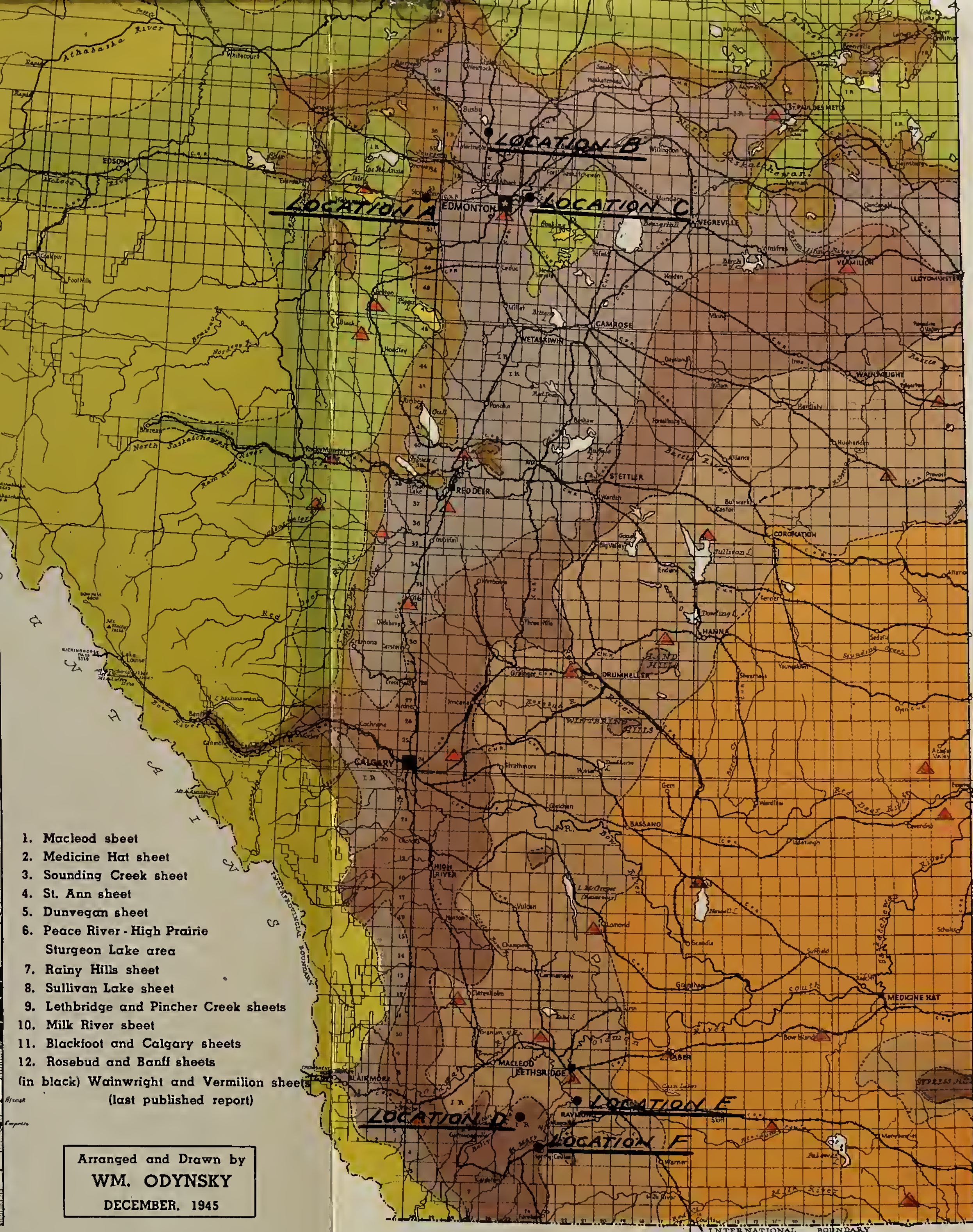
CLIMATE—Annual precipitation averages between 14 and 17 inches. The higher rainfall is in the southern part of the province where there is a correspondingly higher evaporation. Droughts occur only occasionally.

VEGETATION—Grassland in which bluffs of trees are found in places where moisture conditions are more favorable.

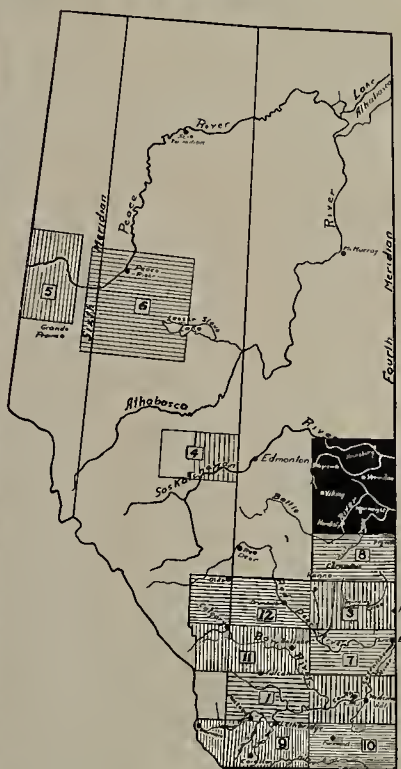
SOIL PROFILE—The normal profile has an A horizon that averages about 10 inches in depth and which in its upper 3 to 6 inches is black in color. The remainder is usually dark brown. The B horizon is usually brown to dark brown and the lime horizon (Bca) is found at depths of 24 to 30 inches below the surface. Generally the depth to the lime layer is considered as indicative of the efficiency of rain penetration.

FERTILITY—Soils in this zone are usually fairly well supplied with nitrogen and organic matter. In any zone exhaustive cropping depletes the soil's native food supply and fibre. A permanent system of cropping provides for the adequate replacement of depleted plant foods and the maintenance of organic matter.

LAND USE—A greater number of soil types can be considered arable than in the brown zones. Wheat is the principal crop grown, but considerably more diversification is possible and should be practised to maintain soil fertility. The non-arable land is generally very good pasture.



LOCATION OF SURVEYED AREAS FOR WHICH SOIL REPORTS AND MAPS HAVE BEEN PUBLISHED



1. Macleod sheet
2. Medicine Hat sheet
3. Sounding Creek sheet
4. St. Ann sheet
5. Dunvegan sheet
6. Peace River - High Prairie Sturgeon Lake area
7. Rainy Hills sheet
8. Sullivan Lake sheet
9. Lethbridge and Pincher Creek sheets
10. Milk River sheet
11. Blackfoot and Calgary sheets
12. Rosebud and Banff sheets
(in black) Wainwright and Vermilion sheets (last published report)

Arranged and Drawn by
WM. ODYNSKY
DECEMBER, 1945

in the black zone.

SOIL PROFILE—Generally quite mixed, ranging from nearly black to gray. The surface horizon consists of a thin layer of semi-decomposed litter (A0) which may be absent in burned over areas, underlain by a mineral horizon that can usually be divided into two parts. The upper (A1) part may be black, gray black or dark brown. The lower part (A2) is frequently somewhat leached of organic matter and considerably grayer than the A1. The total depth of these surface horizons averages about 10 to 12 inches. The B horizons are generally dark brown in color and lime is found at depths of about 30 to 40 inches.

FERTILITY—These soils are usually not as rich as those of the black zone. Leaching of the surface horizons has resulted in the loss of some plant foods.

LAND USE—A system of mixed farming that includes legumes in the crop rotation, supplemented with applications of fertilizer when required, should be practised for best results.

GRAY WOODED

(Boundaries Tentative)

CLIMATE—Annual precipitation averages from about 12 inches in the northern sections to about 20 inches in the southern. This is accompanied by cooler temperatures, lower evaporation and shorter growing seasons than those of the previous zones.

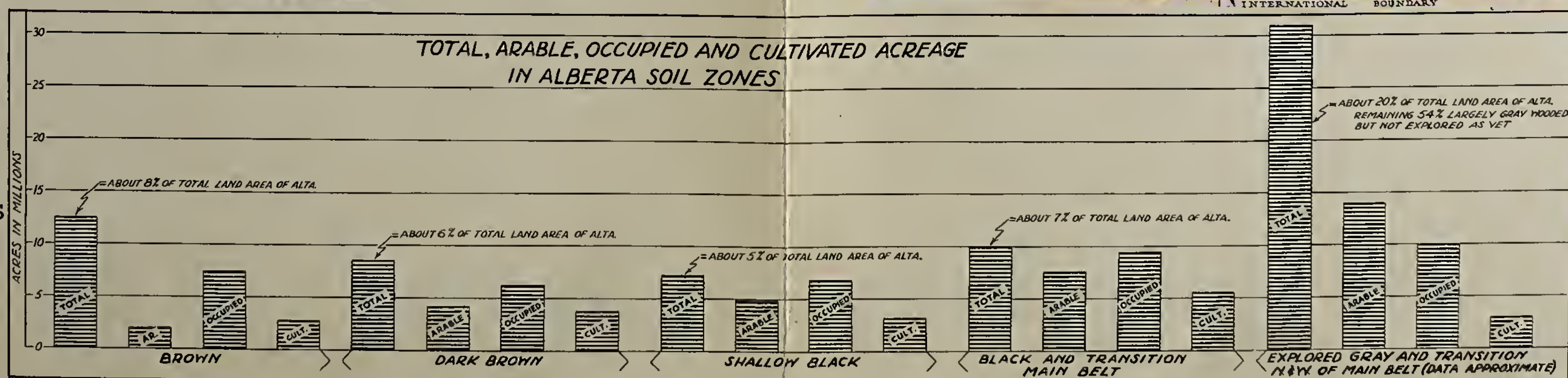
VEGETATION—A mixed deciduous and evergreen woodland in which peats and muskegs frequently occur.

SOIL PROFILE—These soils have developed under humid soil moisture conditions. The surface horizon consists of a semi-decomposed leaf mold layer, A0, that may be absent if the area has been burned over; a thin (sometimes absent) A1 horizon that may be gray black, brown or gray brown, and a severely leached and platy, grayish A2 horizon, whose depth will average about 6 to 8 inches. The B horizons are heavier textured, compact, and often darker in color than the A. The depth to lime is quite variable, often ranging from 30 to 50 inches.

FERTILITY—Soils in this zone are relatively less fertile because of leaching; the deeper the leached layer, the less fertile. However patches of transition soils are found within the zone.

LAND USE—This is a mixed farming area in which legumes, hays and coarse grains are the most desirable crops. Rotations including legumes and supplemented with fertilizers, where needed, have given the most satisfactory results.

AREA NOT EXPLORED
SOIL SURVEYS—BELIEVED GRAY WOOD



LOCATION OF EXPERIMENT STATIONS
SCHOOLS OF AGRICULTURE.

E MAP BY THE DEPARTMENT OF LANDS
AND MINES, EDMONTON, ALBERTA

Lithographed by Hamly, Edmonton

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DEPARTMENT OF EXTENSION
UNIVERSITY OF ALBERTA
Edmonton, Alberta

BRIEF DISCUSSION OF SIGNIFICANCE OF LABORATORY TESTS

1. Specific Gravity

The specific gravity will give a fair indication of the type of soil being tested. Silica sands have a specific gravity of about 2.65, clay soils vary from 2.70 to 2.80. The presence of organic material decreases the specific gravity considerably.

The principal use of the specific gravity is in computations incidental to other soil tests.

2. Mechanical Analysis

A comparison of grain-size curves does indicate great differences between soils. While such curves may be satisfactory for classification purposes for soils of the same geological origin, any general system of classification based on grain-size alone is likely to be misleading, since the physical properties depend on many factors other than the grain-size. However, grain-size distribution curves do indicate the susceptibility of a soil to frost action.

3. Atterberg Limits

The limit values of a soil are required for classification purposes. For sands and silts, the plasticity index may be zero, or even negative. For soils with an appreciable clay content, the plasticity index may be as high as 100. The clay soils of this area generally have plasticity indices ranging from 20 to 50. The compressibility of a clay is approximately directly proportional to its liquid limit, that is, a high liquid limit indicates high compressibility. A great deal of information is provided when the liquid limit and plasticity index are plotted on the Plasticity Chart, Figure 15.

The shrinkage limit of the local soils is particularly

important. A low value of the shrinkage limit indicates large volume changes, and swelling pressures with variations in the moisture content.

4. Proctor Compaction Tests

The optimum moisture content of a soil can be defined as that moisture content at which a given soil may be compacted most efficiently with a given method of compaction. The laboratory compaction test is the basis of control and inspection of the construction of highway embankments.

Since Proctor developed his standard test, the weight of compactive equipment has considerably increased, and accordingly the modified Proctor test is often being used as a basis of control. The modified Proctor density test gives higher densities and lower optimum moisture contents. When the laboratory test is to be used as a control on the field compaction it must be remembered that the results of the test should compare to the results obtained by the field equipment. The available field equipment should govern the choice of the Proctor density test to be used as the basis of control.

5. Triaxial Compression Tests

The triaxial compression test is performed to determine deformation, stress and strength characteristics of a soil. Quick shear tests can be used to determine the shearing resistance of a material to be placed in an embankment. However, the differences between a compacted laboratory sample and material in the embankment are still uncertain. Hence the test results must be used with care.

A series of tests to failure are run on samples at varying lateral pressures. One Mohr circle is plotted for each test, and then a line is drawn tangent to these Mohr circles. This rupture line is tangent to any Mohr circle drawn for failure conditions. Thus any Mohr circle

lying below the rupture line represents safe stress conditions. Any Mohr circle above the rupture line represents failure conditions. The intercept of the rupture line on the shearing stress axis is the cohesion "c", and the slope of the rupture line is the angle of internal friction, "ϕ".

The compressive strength of the soil can be determined by use of the following formula:

$$q_c = p_c (N_\phi - 1) + 2c\sqrt{N_\phi} \quad \ldots\ldots\ldots \quad (1)$$

where,

q_c = confined compressive strength

p_c = confining or lateral pressure

N_ϕ = flow value = $\tan^2 (45^\circ + \frac{\phi}{2})$

This confined compressive strength is equal to the diameter of the Mohr circle, $\sigma_1 - \sigma_3$. Where the confining pressure on the material is zero or where the material is near the surface of the embankment, then the unconfined compressive strength is:

$$q_u = 2c\sqrt{N_\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

The ordinary highway embankment is usually not more than ten feet in depth; and then the confining pressure is not more than $\frac{1}{2}$ ton per square foot, and probably much less.

Thus it is recommended that the compressive strength of a material in a highway embankment be determined by use of the relation, (2), above.

6. California Bearing Ratio Tests

The California Bearing Ratio, (C.B.R.), test is the basis of the California State Highway Department method of design of flexible

pavements. The C.B.R. test, which is a penetration type test, determines a modulus of shearing resistance of soils. This modulus value is then used in empirical design curves to determine the total thickness of base and surface to protect the subgrade against shear failure.

The validity of the laboratory C.B.R. test results are dependent on preparation of the sample to duplicate field conditions. At the present time, there is considerable doubt as to whether the laboratory compacted samples have physical properties similar to the material in place during or after construction. Also the test is considered valid only when a large portion of the deformation under penetration is shear deformation.

The U. S. Waterways Experiment Station has recently completed a laboratory investigation of the California Bearing Ratio test and its applications. The results of this investigation are found in the manual, "The California Bearing Ratio Test as Applied to the Design of Flexible Pavements for Airports". A synopsis of the results shows that:

(1) Wide variations in the C.B.R. test results on laboratory compacted samples resulted. These variations are largely due to the method of preparation of the sample.

(2) It is not known how closely the physical properties of laboratory compacted samples correspond to those obtained by the various field compaction methods now available. Thus it will be necessary to study the physical properties of field compacted soils.

The Department of Transport has recently completed an airport evaluation program. The results of this investigation conclusively prove that the C.B.R. method of design is too conservative for the design of airport runways and taxi-strips.

7. Freezing Tests

Freezing tests are conducted on laboratory samples for the purpose of determining the volume change that is likely to occur in the material when in place in the embankment. If the void ratio is high, the expansion due to the water freezing is likely to occur in the voids. If the void ratio is small, then all of the expansion cannot be taken up by the voids and, accordingly, the soil mass will increase in volume. This increase in volume can be very detrimental. No attempt has been made in this study to measure the effects of frost action in either the topsoils or subsoils. As previously mentioned, little is known of the relationship between the physical properties of compacted laboratory samples and field compacted material.

8. Consolidation Tests

The results of the consolidation test are used to compute the settlement of structures on compressible clays. From the void ratio-pressure curve, the compressibility of the soil may be determined. The results of the consolidation test have little use in highway engineering. However, the time curve does permit a computation of the coefficient of permeability of compacted samples. This coefficient of permeability is necessary to investigate the effect of frost on soils.

II

PROCEDURE

For the purposes of this laboratory study, it was decided that approximately one hundred pounds of both the topsoil and subsoil from each location would be required.

A system of identifying the various soils was adopted as follows: the first letter in the soil number was used to identify the location, the second letter to identify the type of soil, and the third number to identify the number of samples of each soil received. For example, the samples received from location "C" would be identified as follows:

C-T-1, the first sample of topsoil received from location "C".

C-L-1, the first sample of loam soil received from location "C", (the loam soil being the layer of brown soil directly below the blacker topsoil).

C-S-1, the first sample of clay soil or subsoil received from location "C".

After identifying each soil, the material was thoroughly mixed and air-dried, after which the sample was prepared. Preparation of the sample involved breaking down of the soil lumps, until the material would pass a $\frac{1}{4}$-inch sieve. Material such as rocks and roots, not passing the $\frac{1}{4}$-inch sieve was discarded, since the A.S.T.M. requires that only material passing a $\frac{1}{4}$-inch sieve shall be used in Proctor density tests. For each sample so prepared, it was observed that not more than two per cent of the material had to be discarded.

The following tests were performed on each soil type, with the noted exceptions, according to the test procedures outlined by, "Notes on Soil Testing for Engineering Purposes", by A. Casagrande and R. E. Fodum, 1940.

1. Two specific gravity determinations
2. Atterberg limit tests
3. Grain size analyses on A-T-1, B-T-1, B-S-1
4. Standard and Modified Proctor density tests on A-T-1, A-S-1, B-T-1, B-S-1.
5. Modified Proctor density tests on the loam soil, C-S-1, and the topsoil, F-T-1.
6. Triaxial compression tests at lateral pressures of 0, 15 and 30 lbs. per sq. in., on samples, A-T-1, A-S-1, B-T-1, and B-S-1, compacted at approximately optimum moisture content for both the standard and modified Proctor density tests.
7. (a) California Bearing Ratio tests on the topsoil, B-T-1, compacted at approximately optimum moisture content for both the standard and modified Proctor density tests.

   (b) California Bearing Ratio tests on the subsoil, B-S-1, compacted at approximately optimum moisture content for the standard Proctor density test.

(Since the C.B.R. test molds were used for these tests, it was necessary to alter the number of blows per layer and number of layers from the A.S.T.M. standards to obtain maximum density. The results obtained are shown in Figures 35 and 36.)

8. (a) Freezing tests on the topsoil, B-T-1, compacted at approximately optimum moisture content in C.B.R. test molds for both the standard and modified Proctor density tests.

   (b) Freezing tests on the subsoil, B-S-1, compacted at approximately optimum moisture content in the C.B.R. test mold for the standard Proctor density test.

   (c) Freezing tests on the soaked samples from the C.B.R. test for the subsoil, B-S-1, compacted at approximately optimum moisture

content for the standard Proctor density test.

(d) Freezing tests on the soaked samples from the C.B.R. test for the topsoil, B-T-1, compacted at approximately optimum moisture content for both the standard and modified Proctor density tests.

(In conducting these freezing tests, the sample in the C.B.R. test molds was placed in the frame ordinarily used in soaking the sample in the C.B.R. test. The open end of the sample was then covered with paraffin to prevent the loss of moisture during freezing. The sample was then placed in the freezing cabinet, with an Ames dial in position, to record the expansion of the compacted soil sample.)

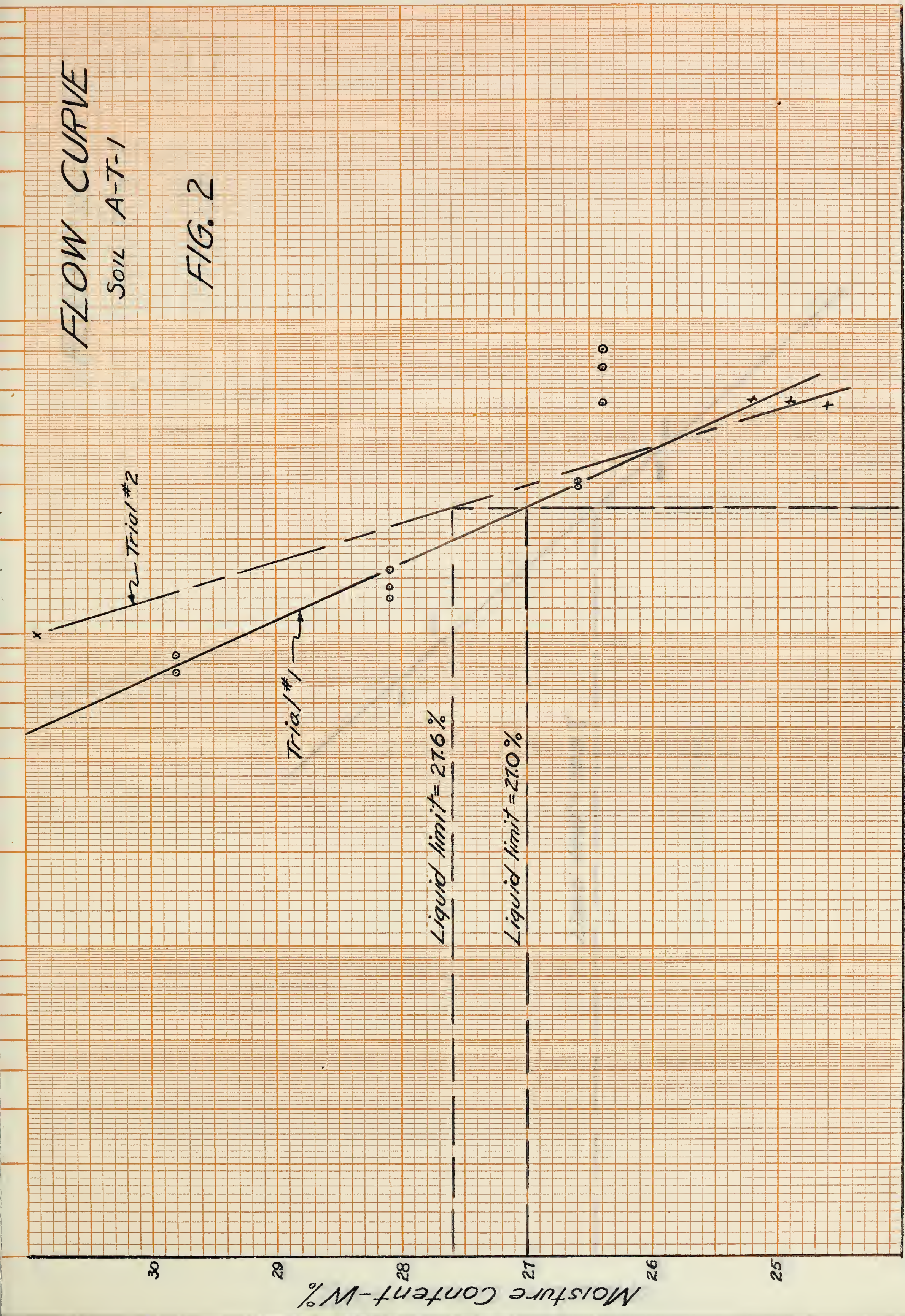
9. Unconfined compression tests on specimens approximately 1 cm. in diameter, obtained from a soaked sample, B-T-1, compacted approximately at optimum moisture content in C.B.R. test molds for both the standard and modified Proctor density tests. These specimens were allowed to partly dry, before performing the compression tests.

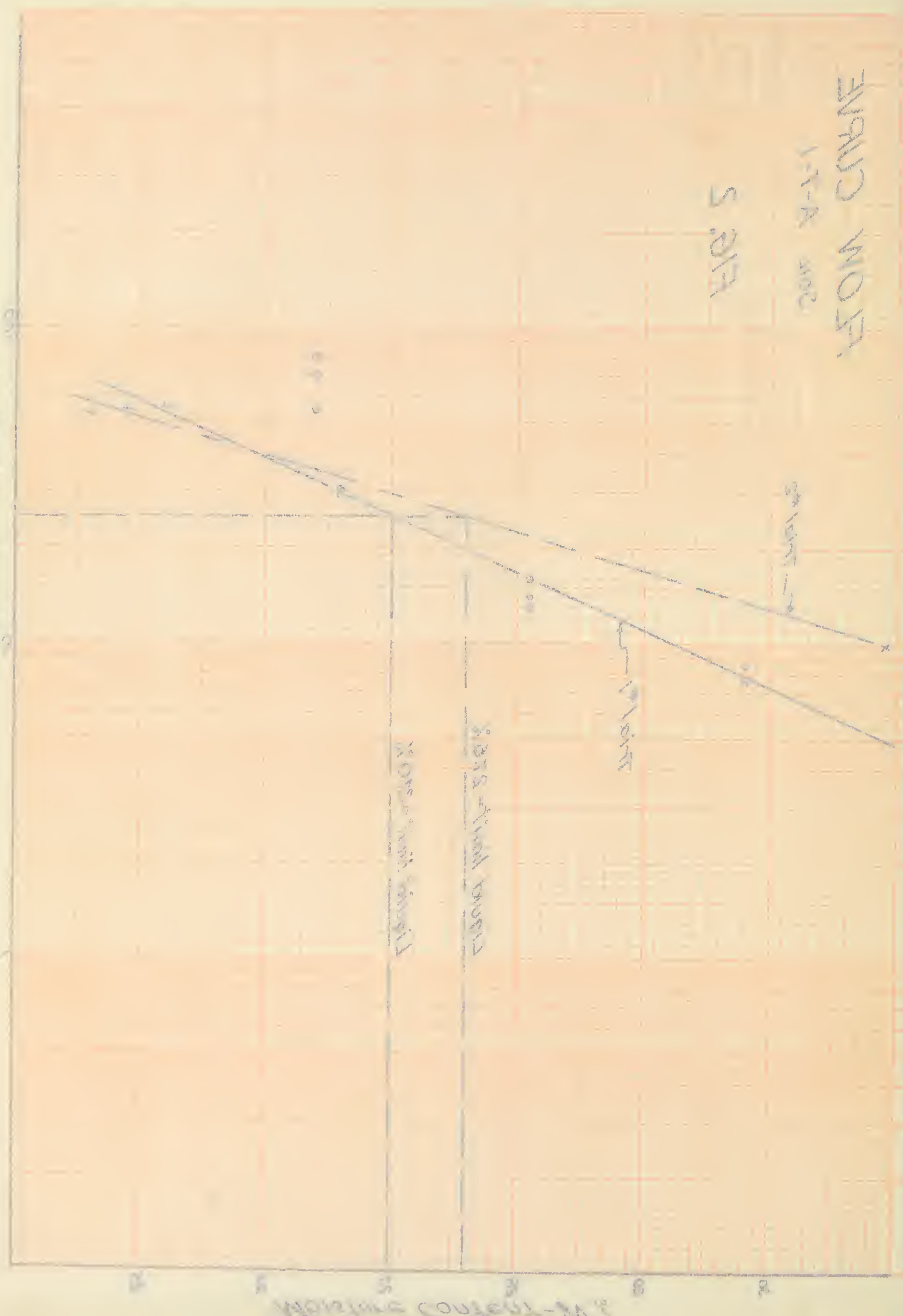
10. (a) Consolidation tests on the topsoil, B-T-1, compacted at approximately optimum moisture content in Proctor density test molds for both the standard and modified Proctor density tests.

(b) Consolidation tests on the subsoil, B-S-1, compacted at approximately optimum moisture content in a Proctor density test mold for the standard Proctor density test.

III

R E S U L T S


FLOW CURVE
SOIL A-T-1
FIG. 2
Trial #2
Trial #1
Liquid limit = 27.6%
Liquid limit = 27.0%
Number of Blows
1
10
100
Moisture Content-W%
30
29
28
27
26
25


FLOW CURVE
Soil A-7-1
FIG. 2
Trial #1
Trial #2
Liquid limit = 27.6%
Liquid limit = 27.0%
Number of Blows
Moisture Content

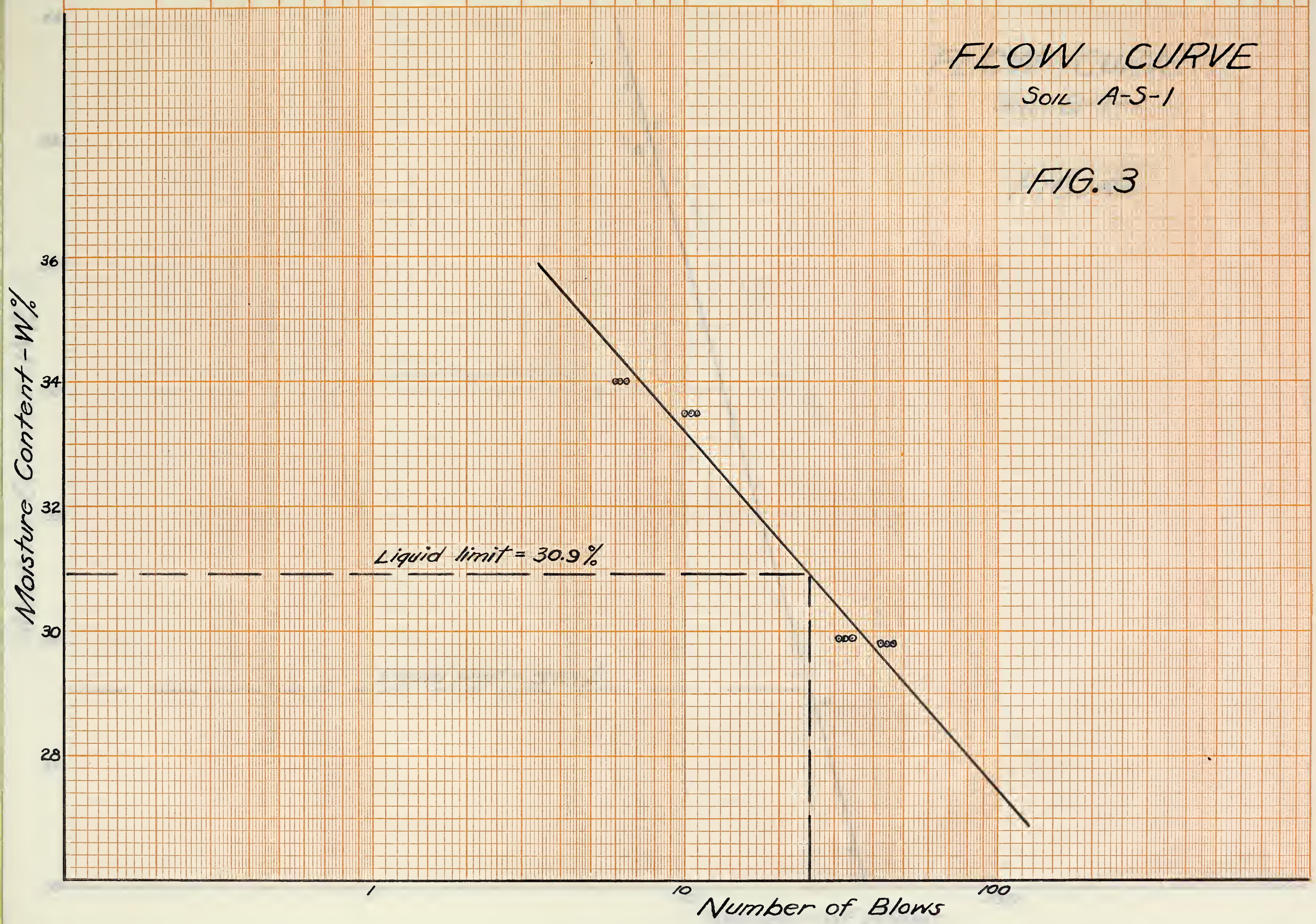
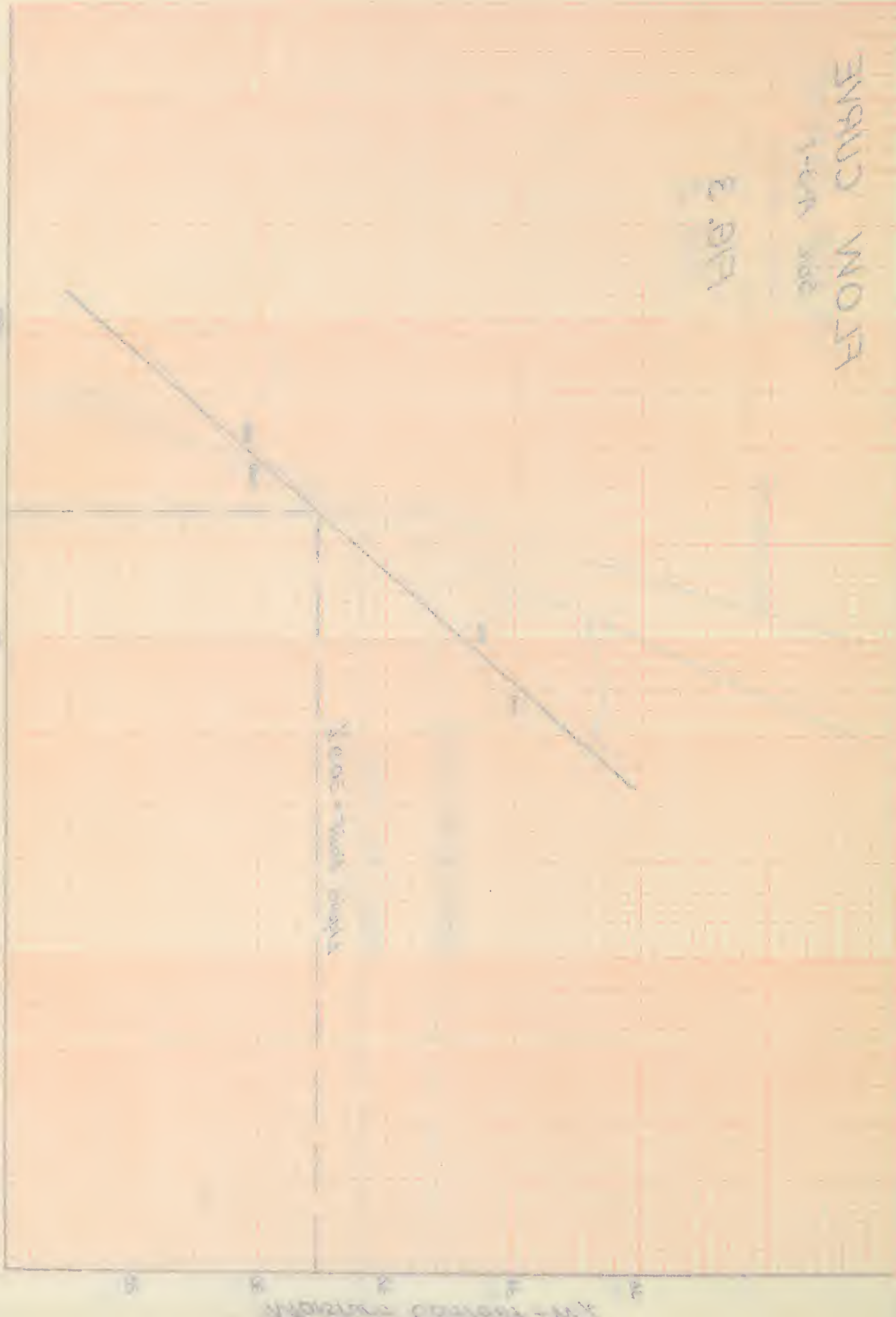

FLOW CURVE
SOIL A-S-1
Liquid limit = 30.9%
Moisture Content - W%
36
34
32
30
28
1
10
100
Number of Blows

FIG. 3



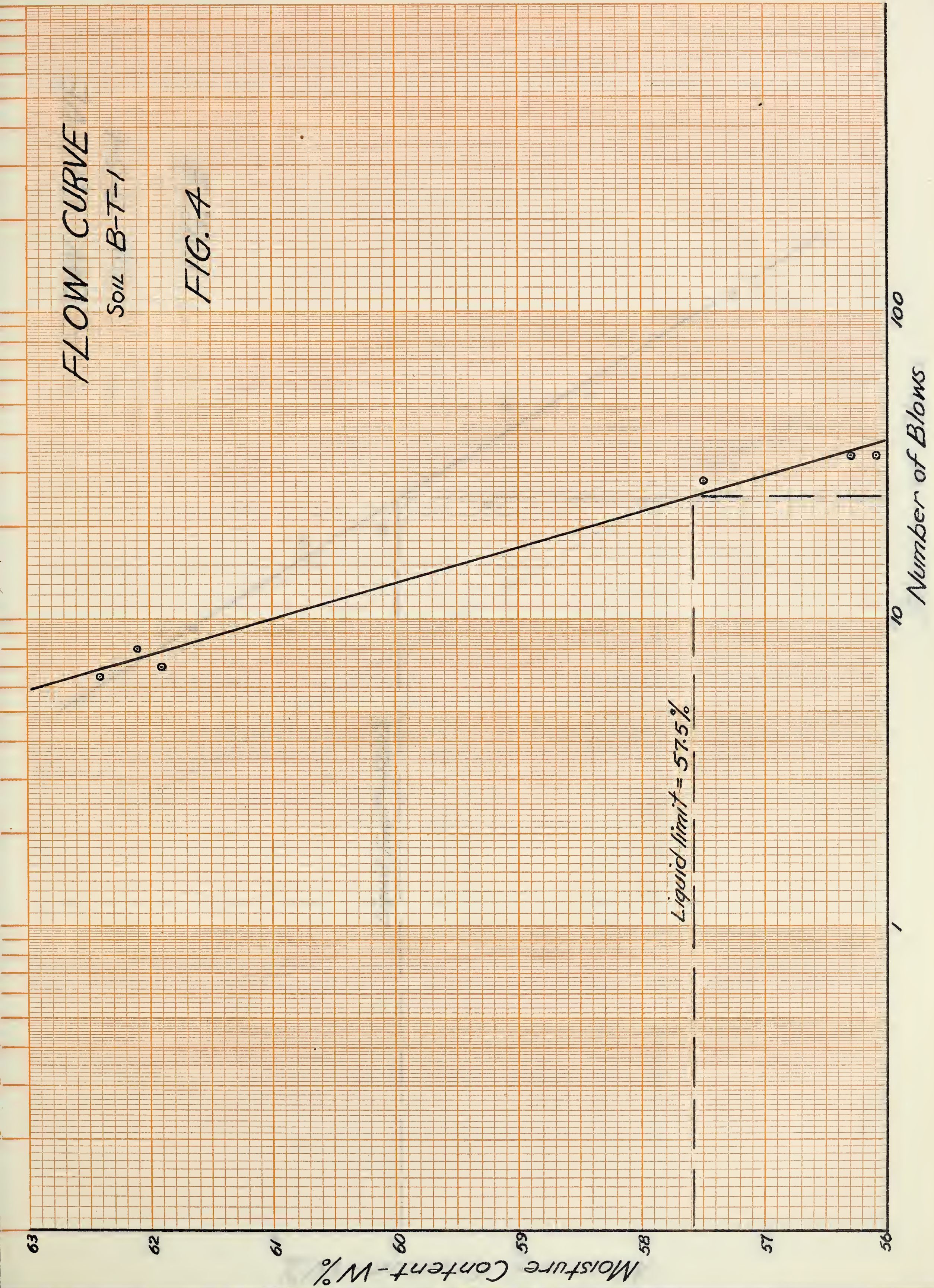
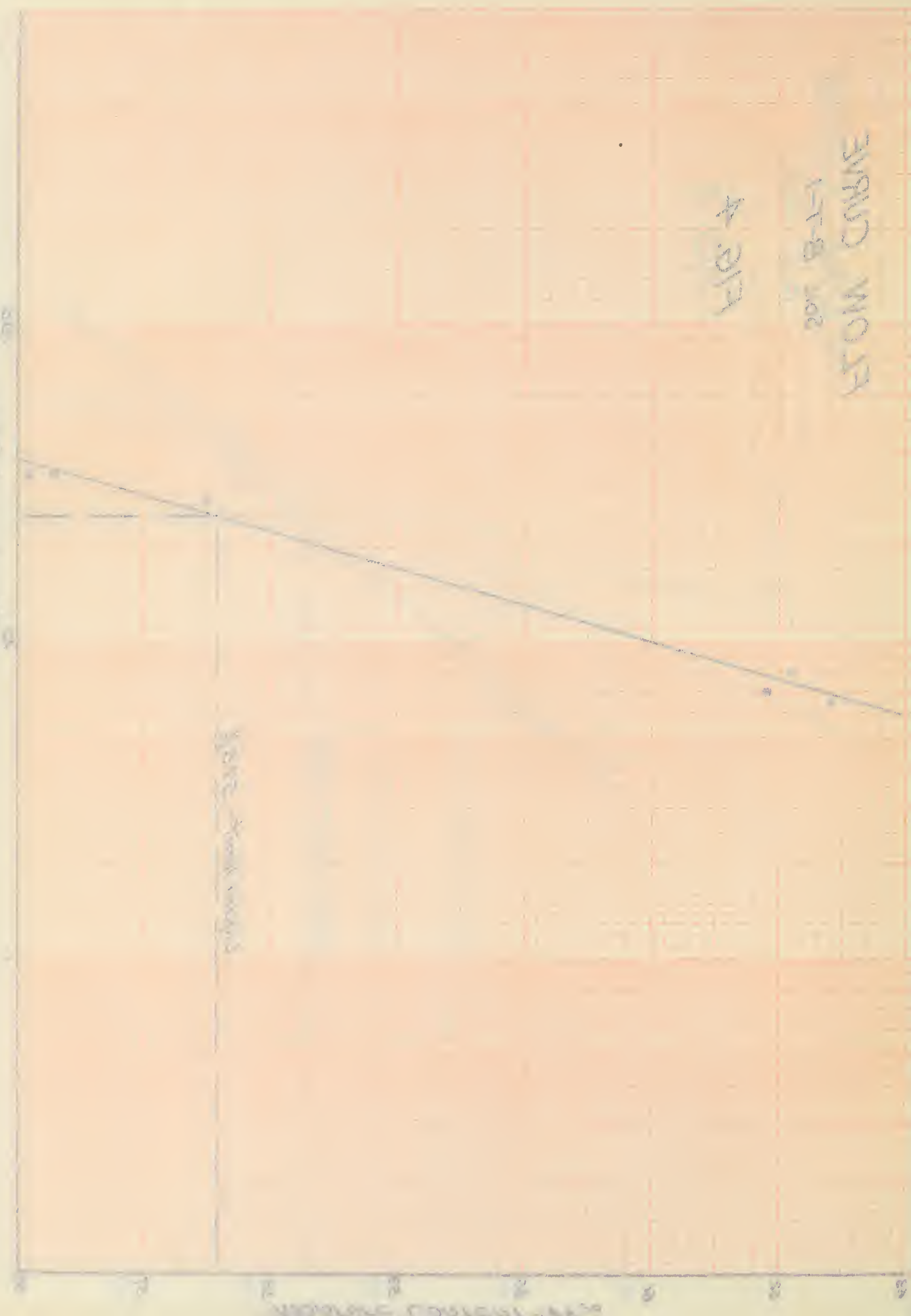

FLOW CURVE
Soil B-T-1
Number of Blows
1
10
100
Moisture Content - W%
63
62
61
60
59
58
57
56
Liquid limit = 57.5%

FIG. 4



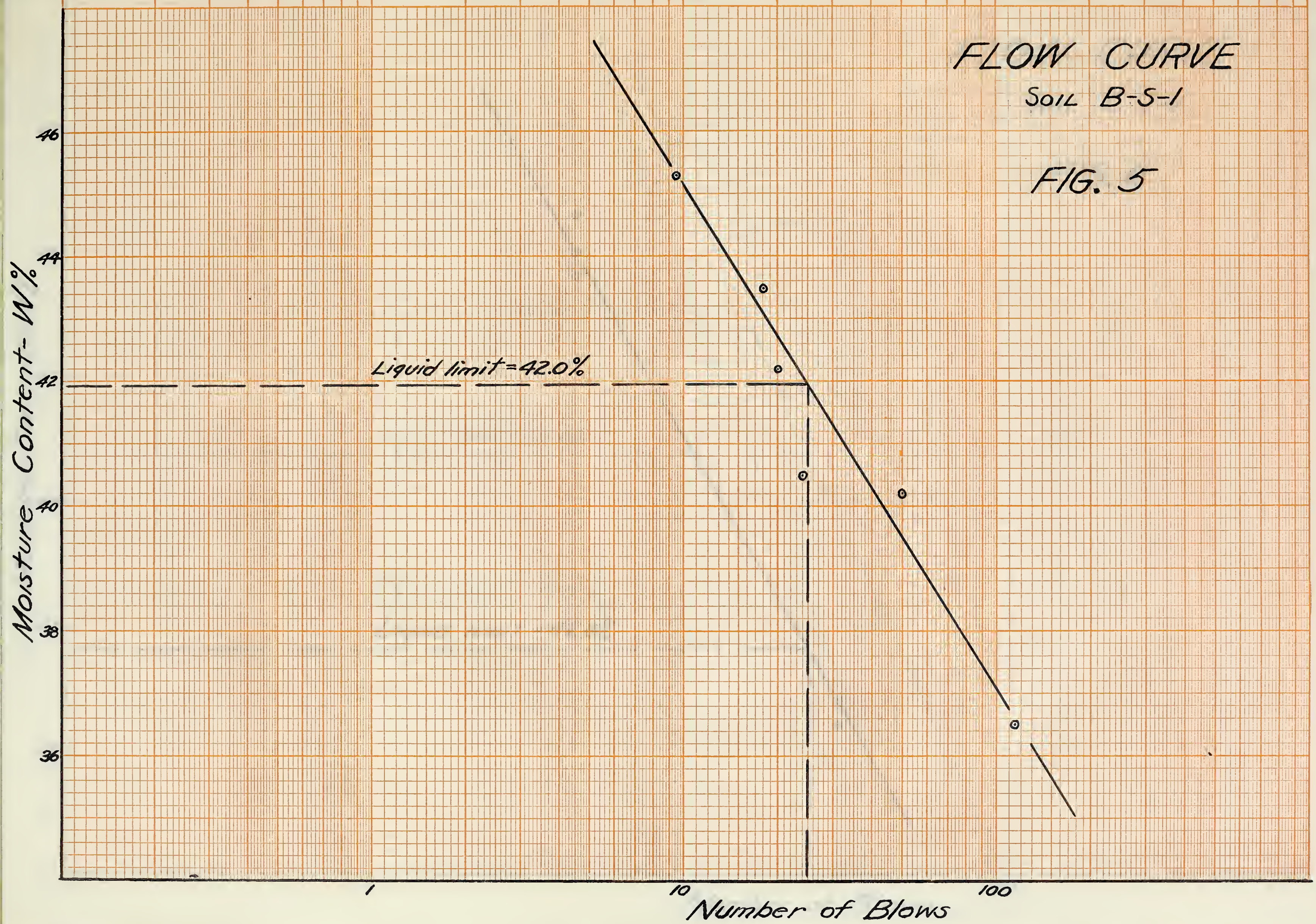
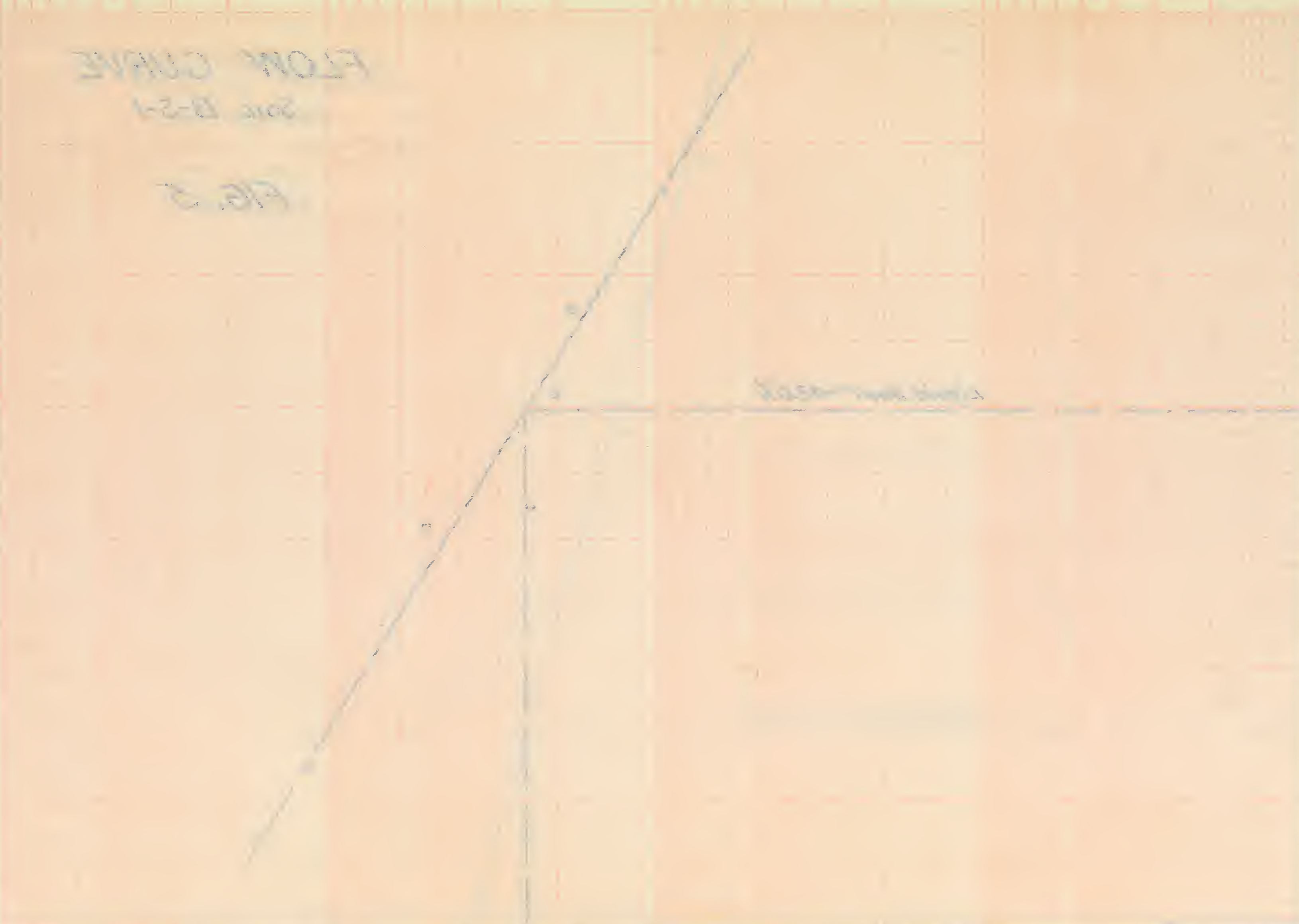
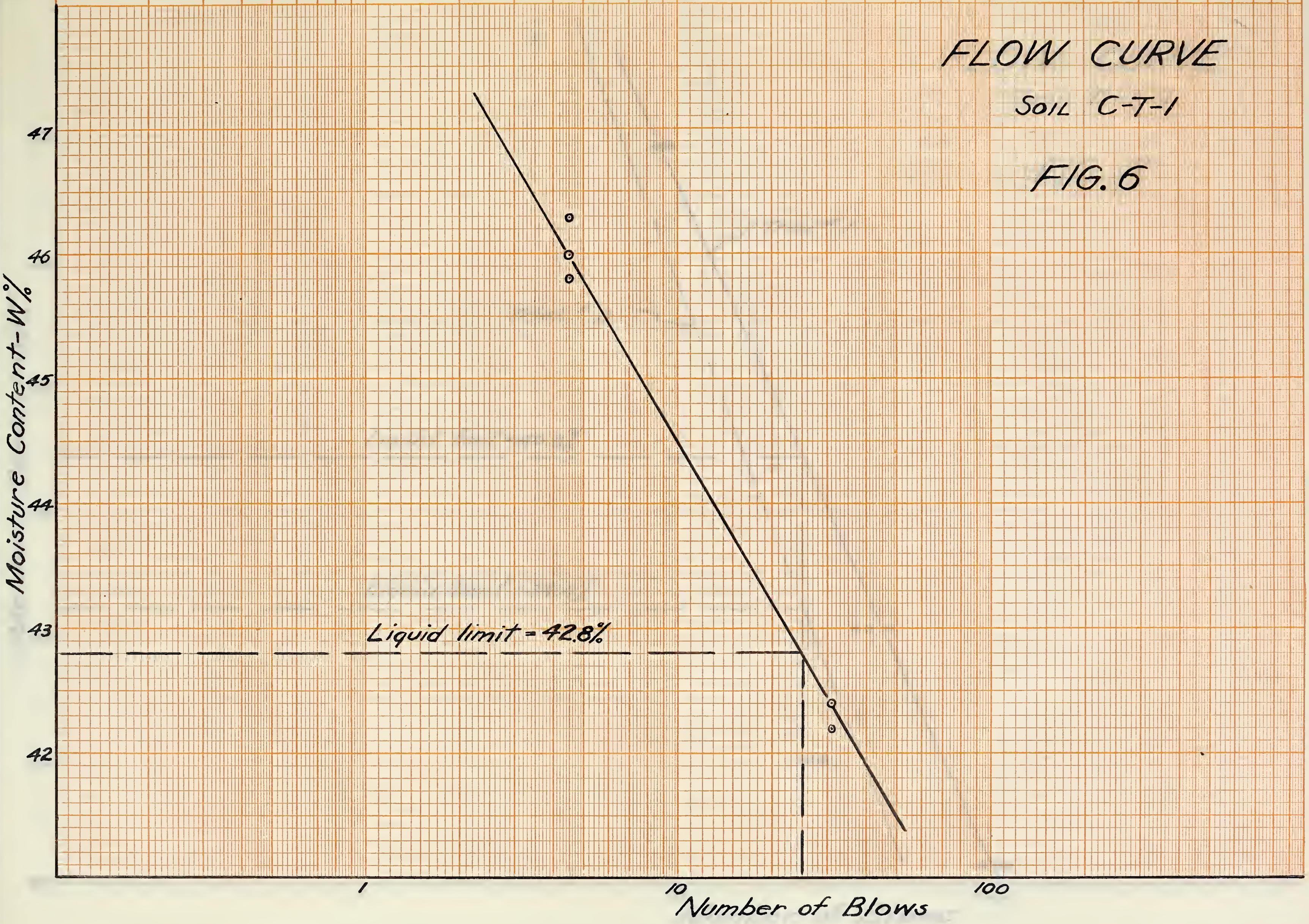
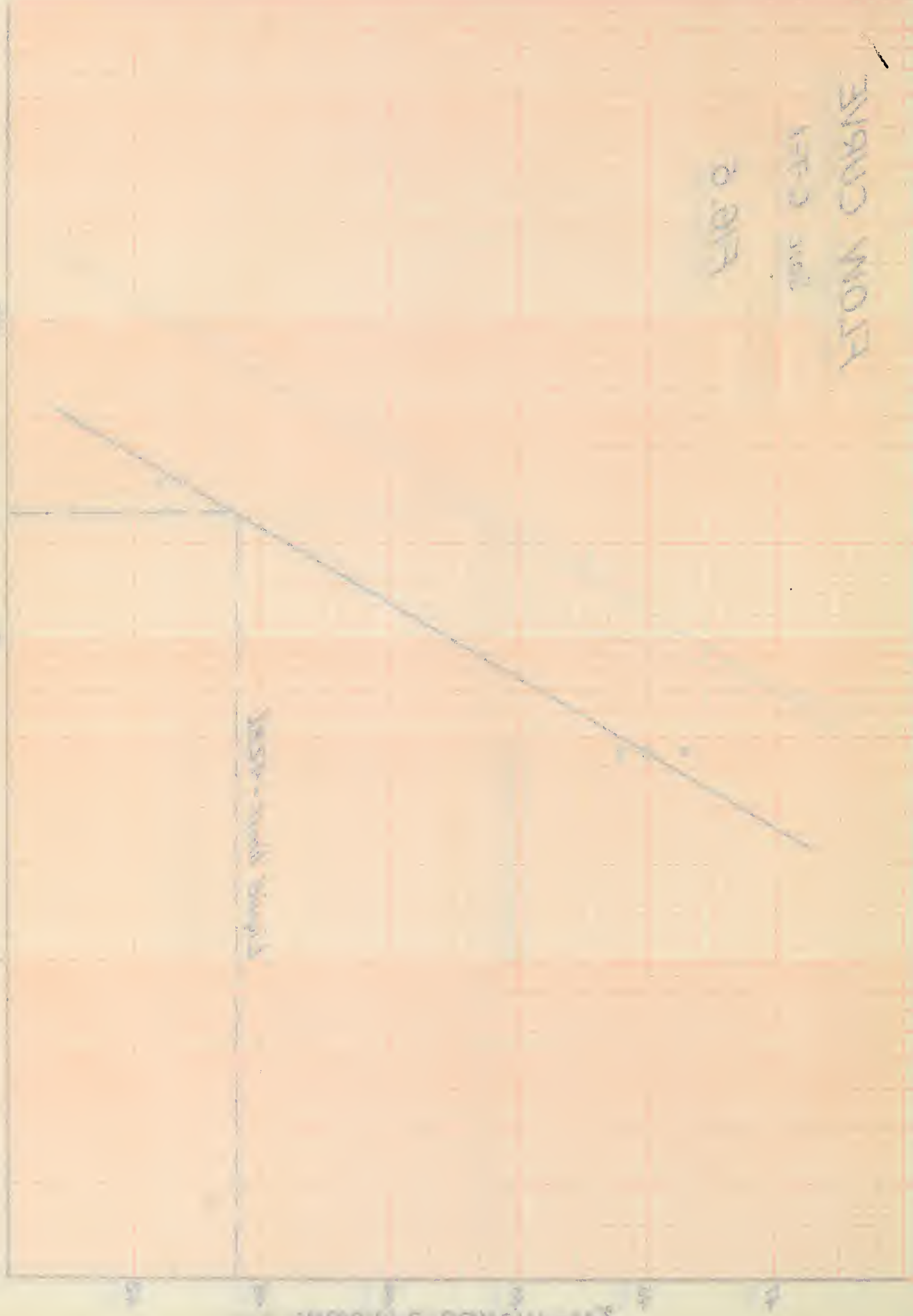

FLOW CURVE
SOIL B-S-1
Liquid limit = 42.0%
Moisture Content- W%
46
44
42
40
38
36
1
10
100
Number of Blows

FIG. 5


FLOW CURVE
Soil B-5-1
FIG. 5

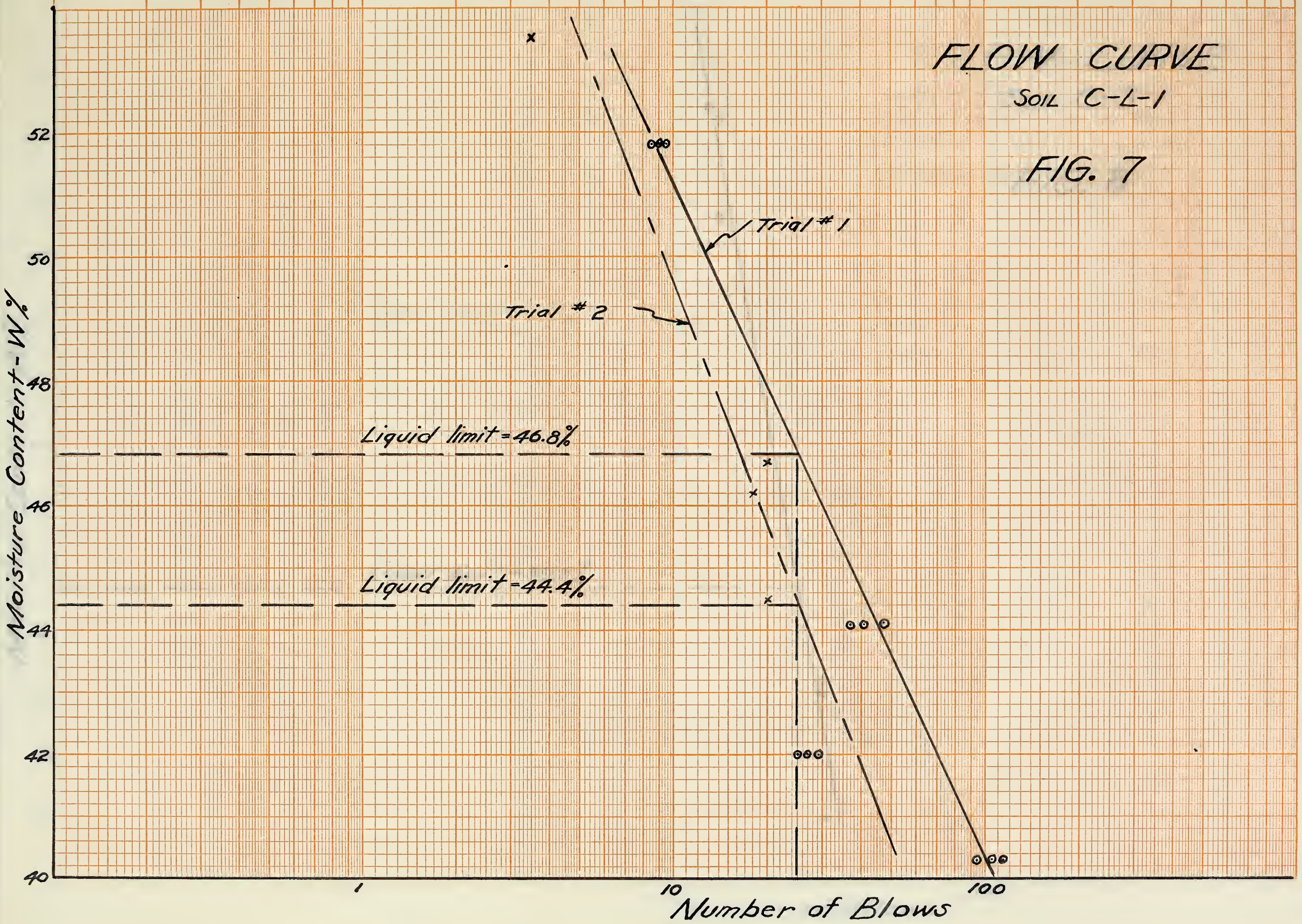

FLOW CURVE
SOIL C-T-1
FIG. 6
47
46
45
44
43
42
Moisture Content-W%
Liquid limit = 42.8%
1
10
100
Number of Blows

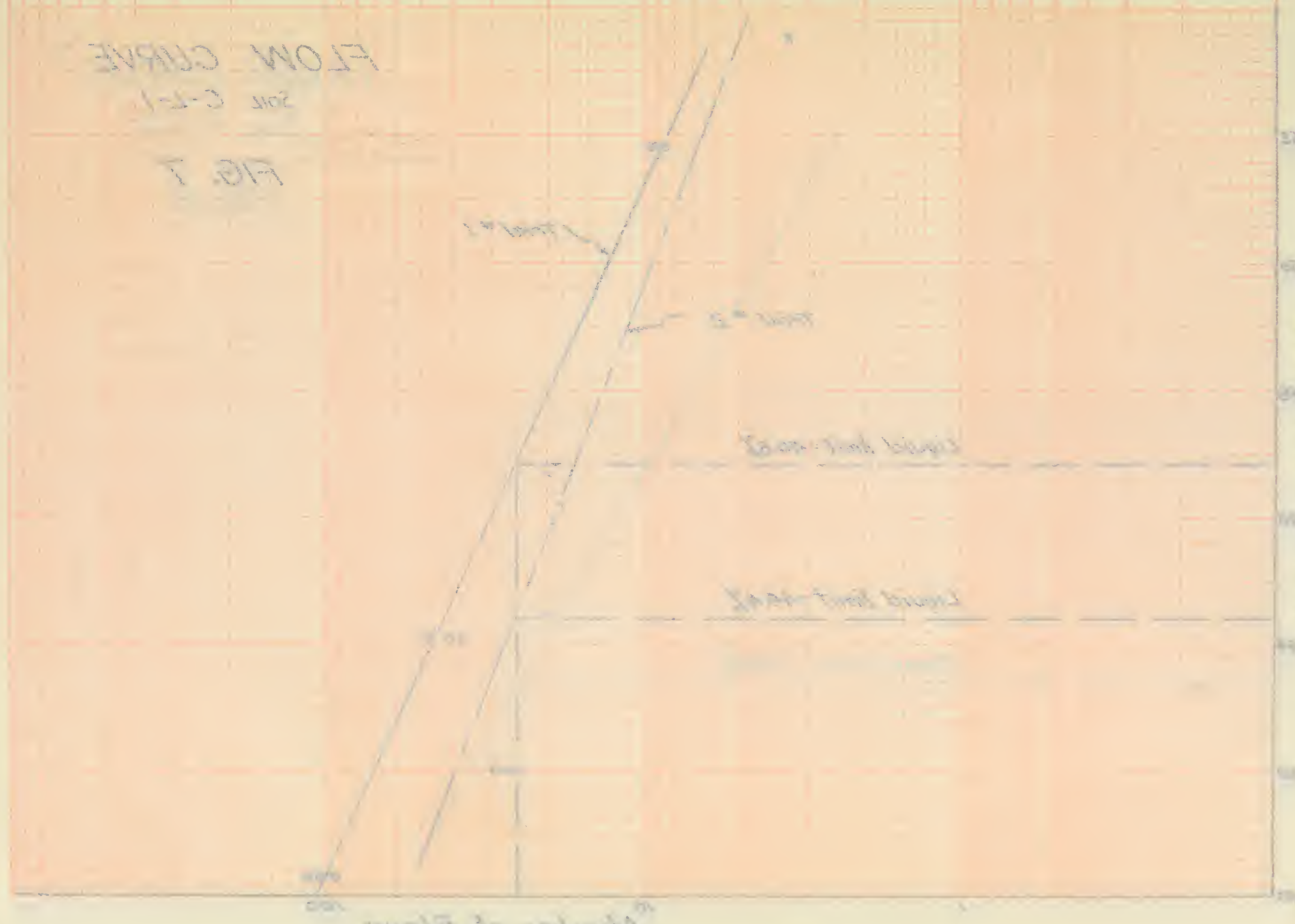


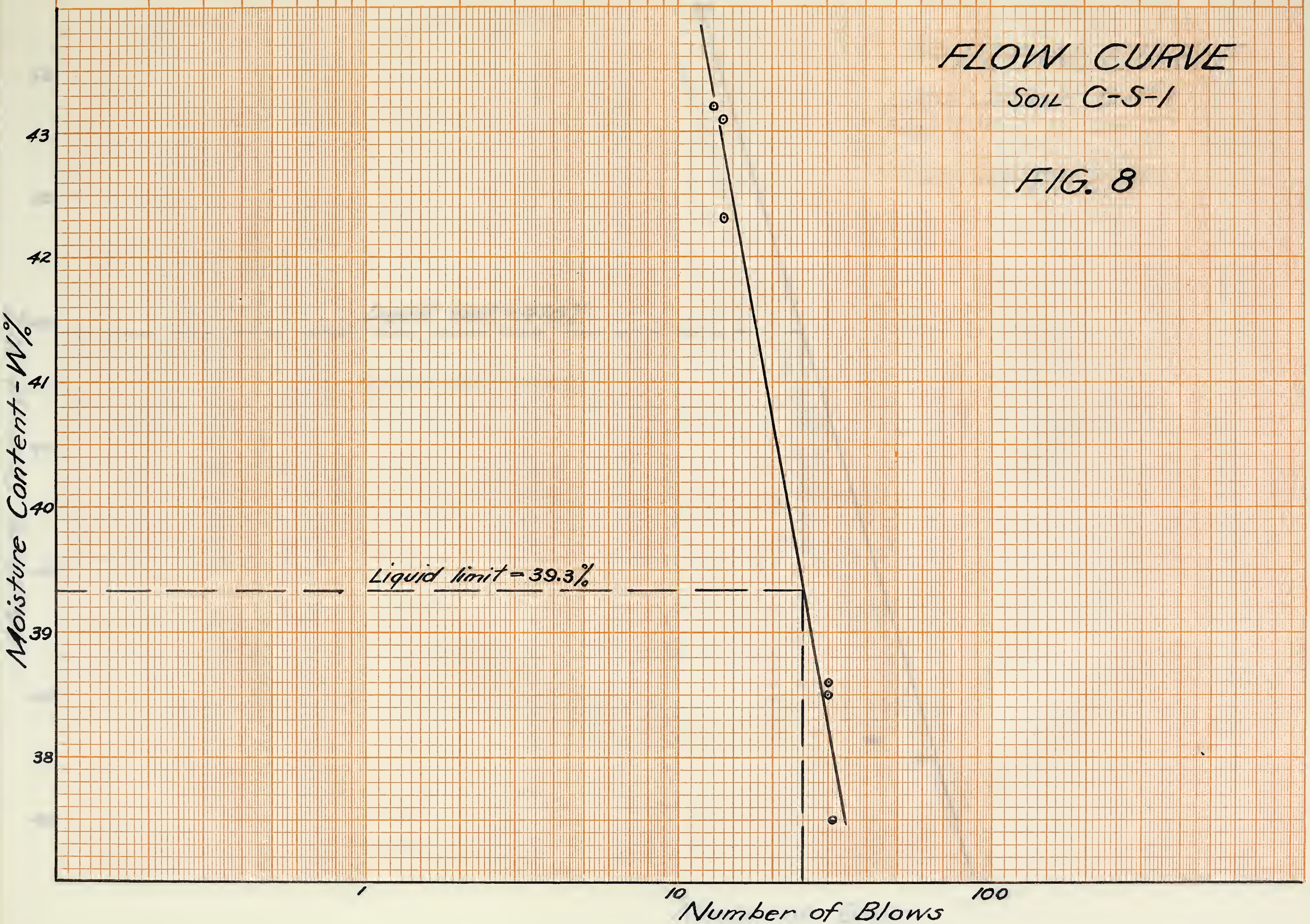
FLOW CURVE

SOIL C-L-1

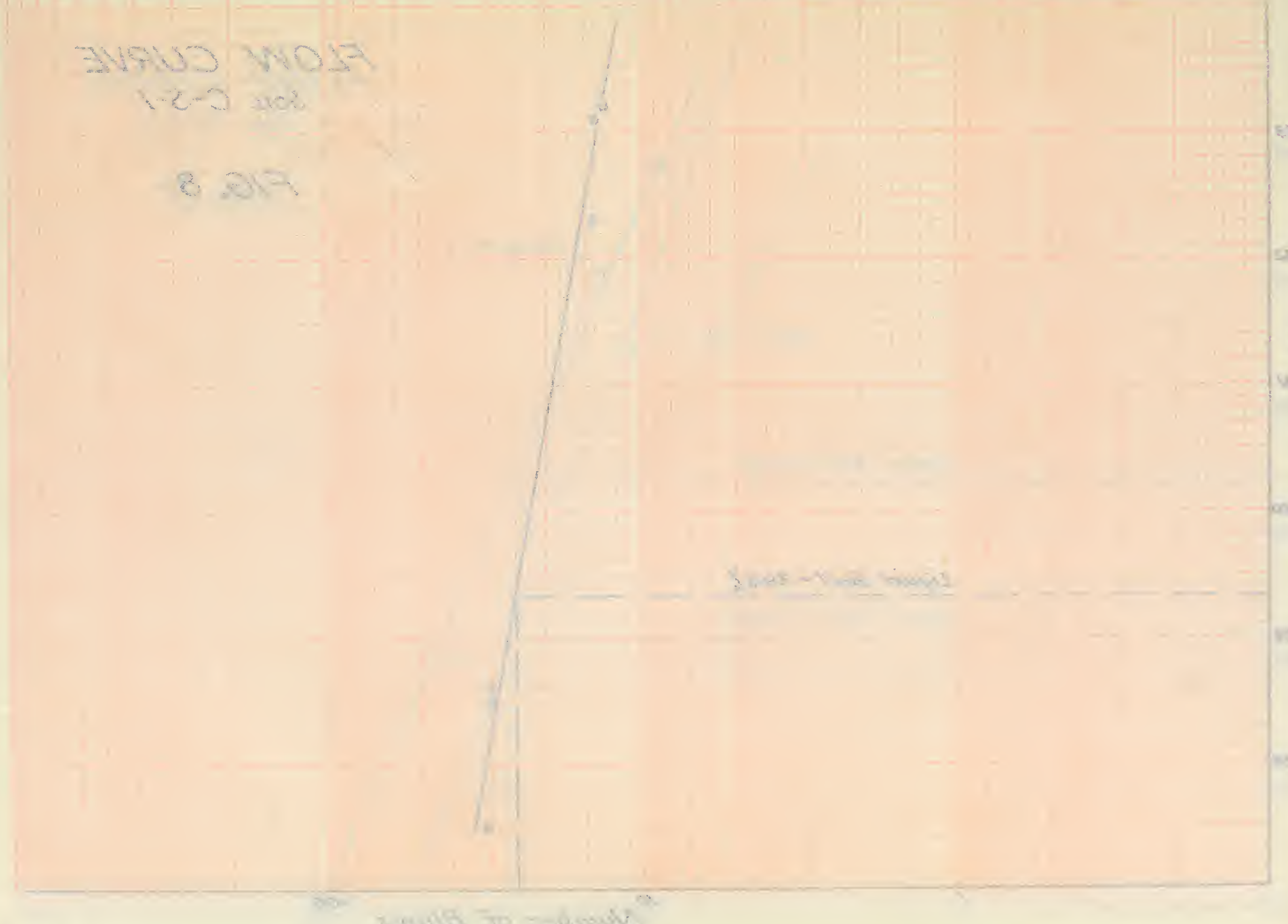
FIG. 7

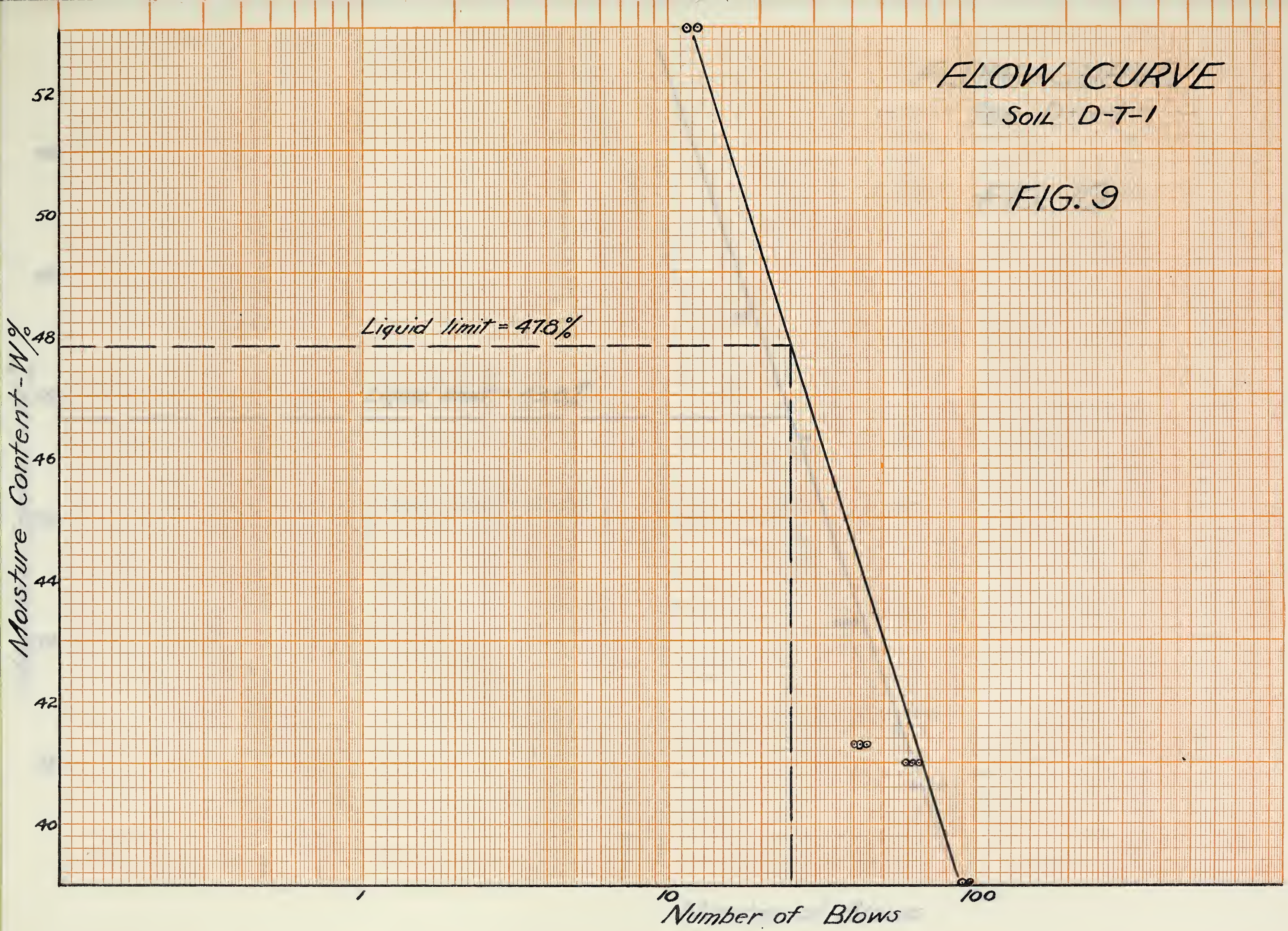






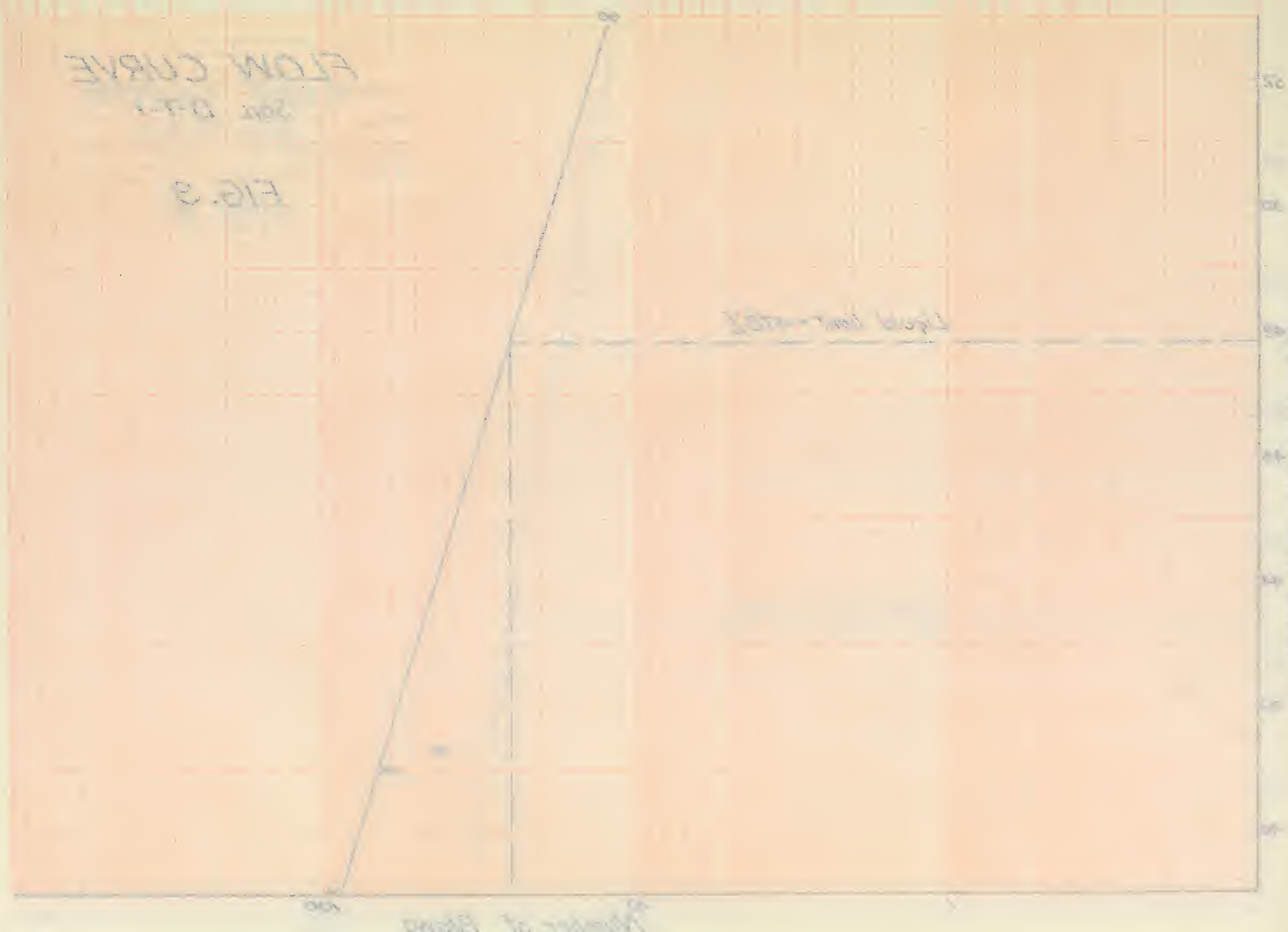
FLOW CURVE
SOIL C-S-1
FIG. 8

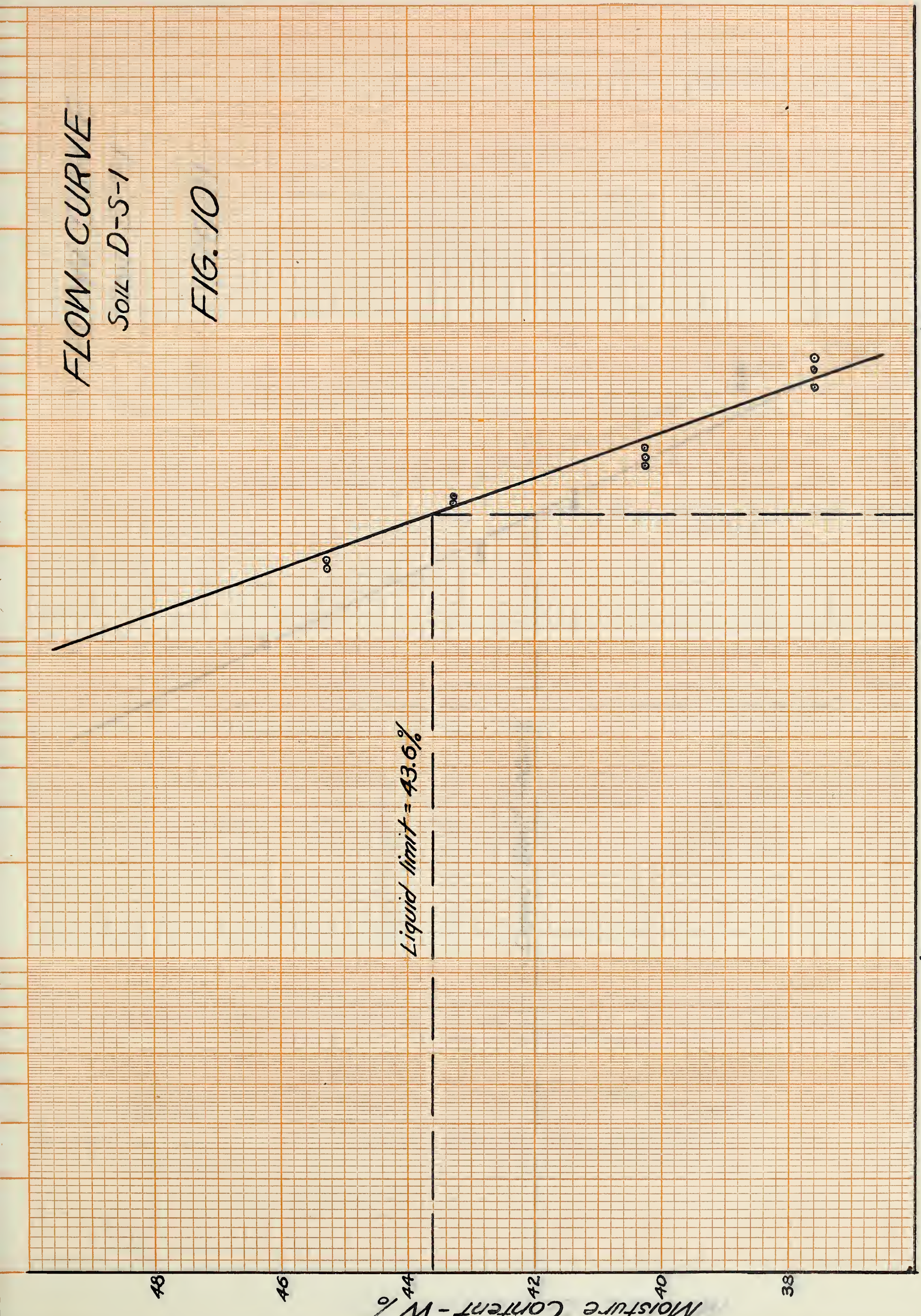

FLOW CURVE
Soil C-5-1
FIG. 8
Number of Blows



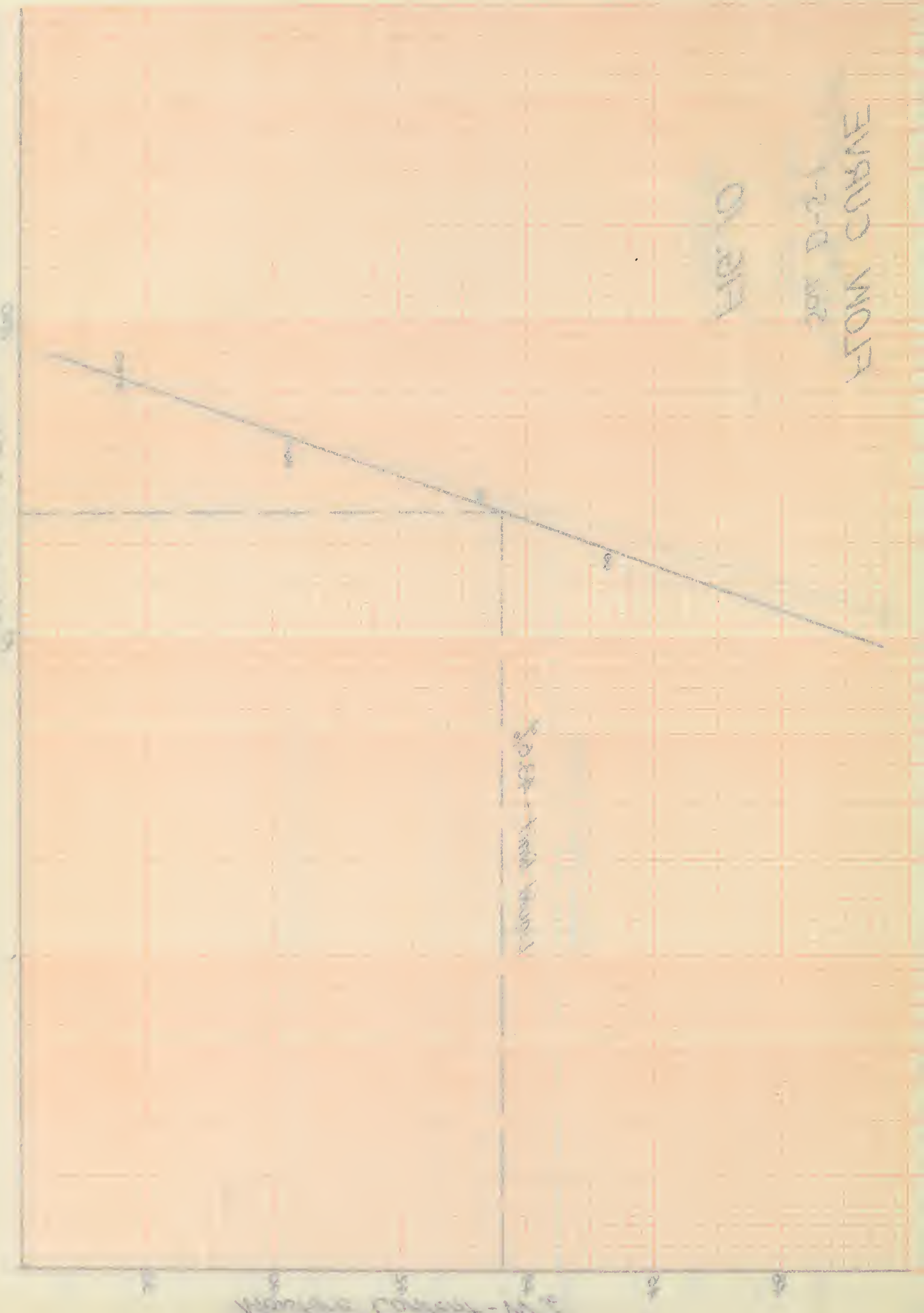
FLOW CURVE
SOIL D-T-1

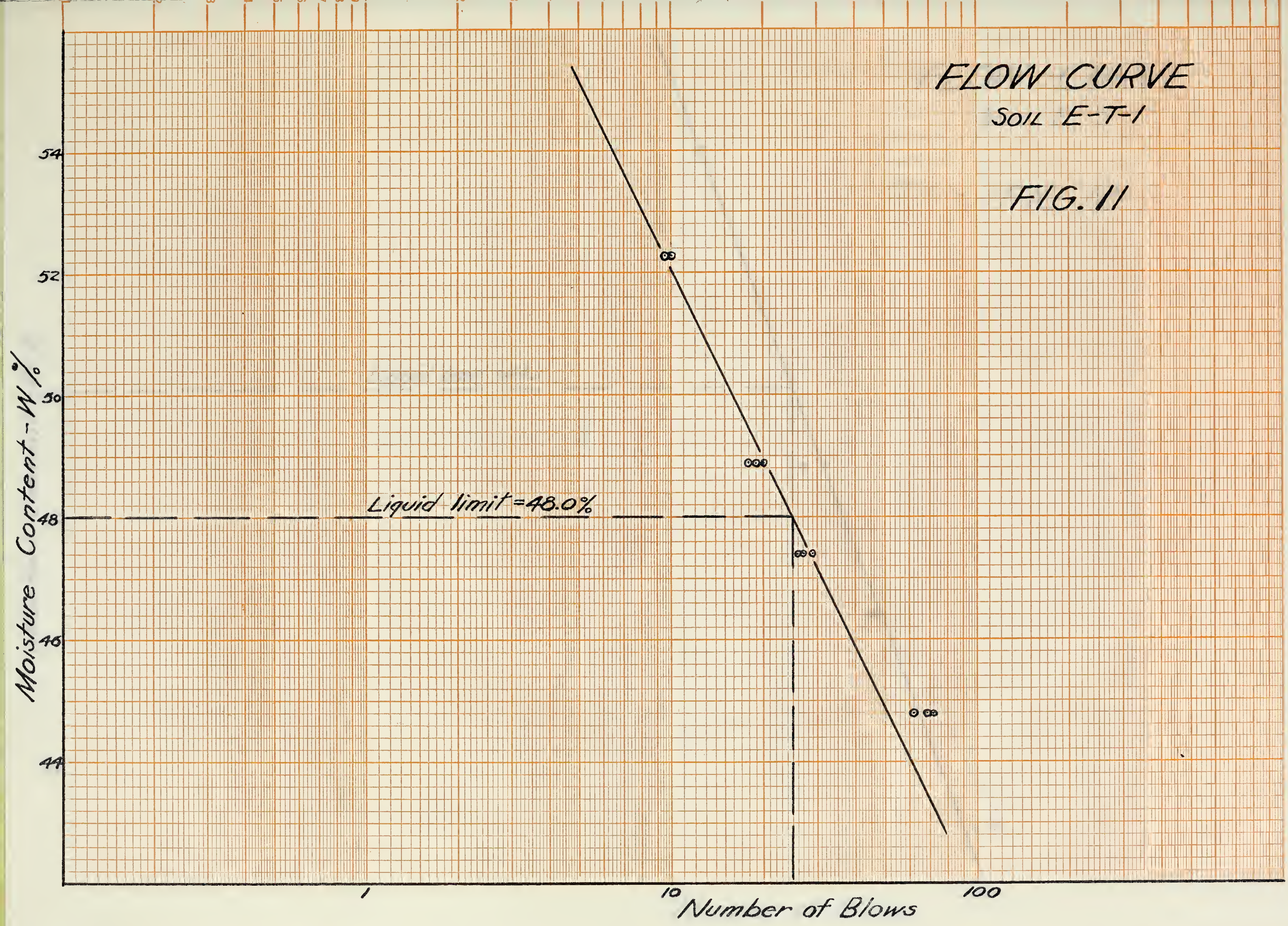
FIG. 9


FLOW CURVE
FIG. 9



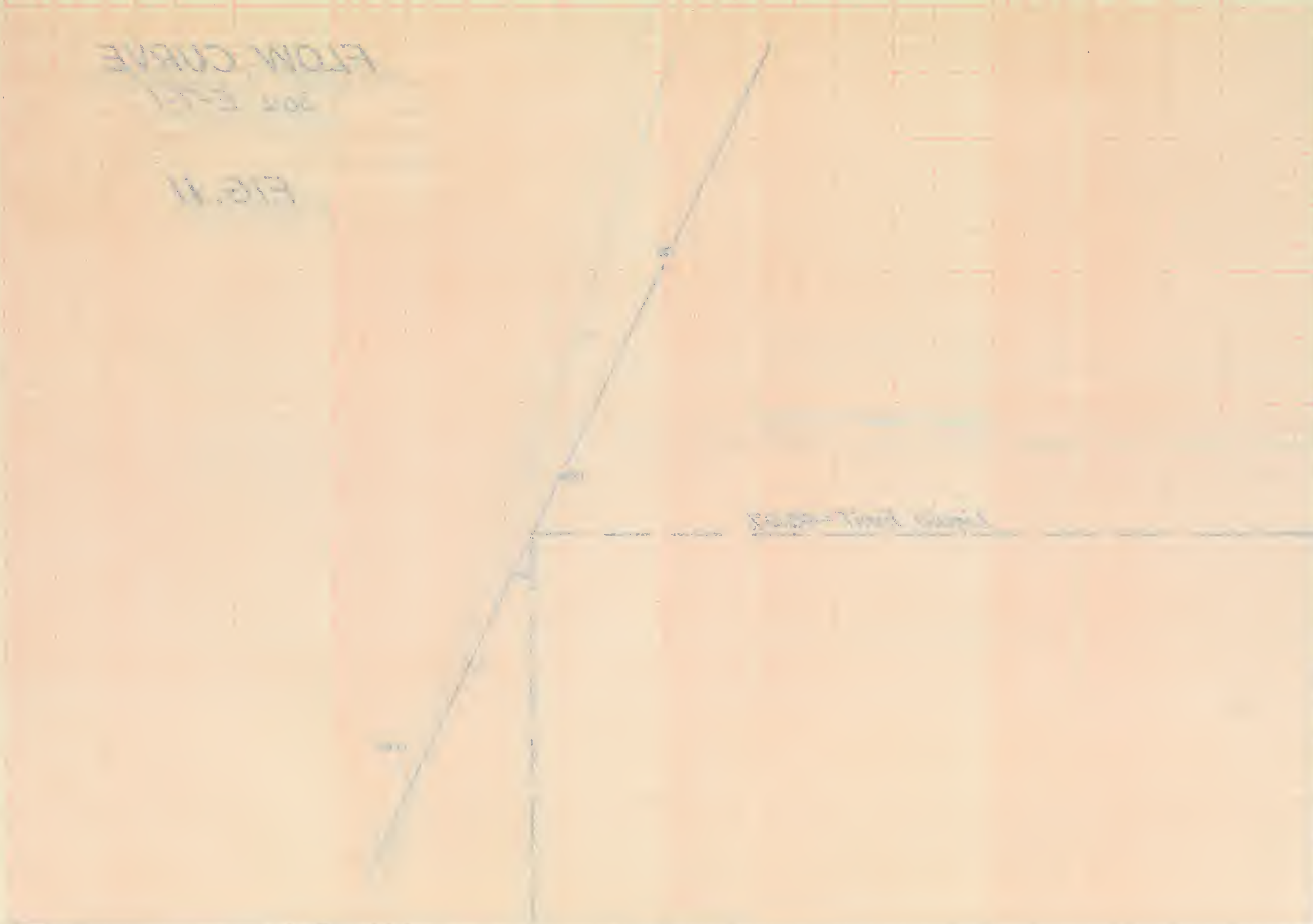
FLOW CURVE
Soil D-S-1
FIG. 10
Liquid limit = 43.6%
Number of Blows
100
10
1
48
46
44
42
40
38
Moisture Content - W %





FLOW CURVE
Soil E-T-1

FIG. 11



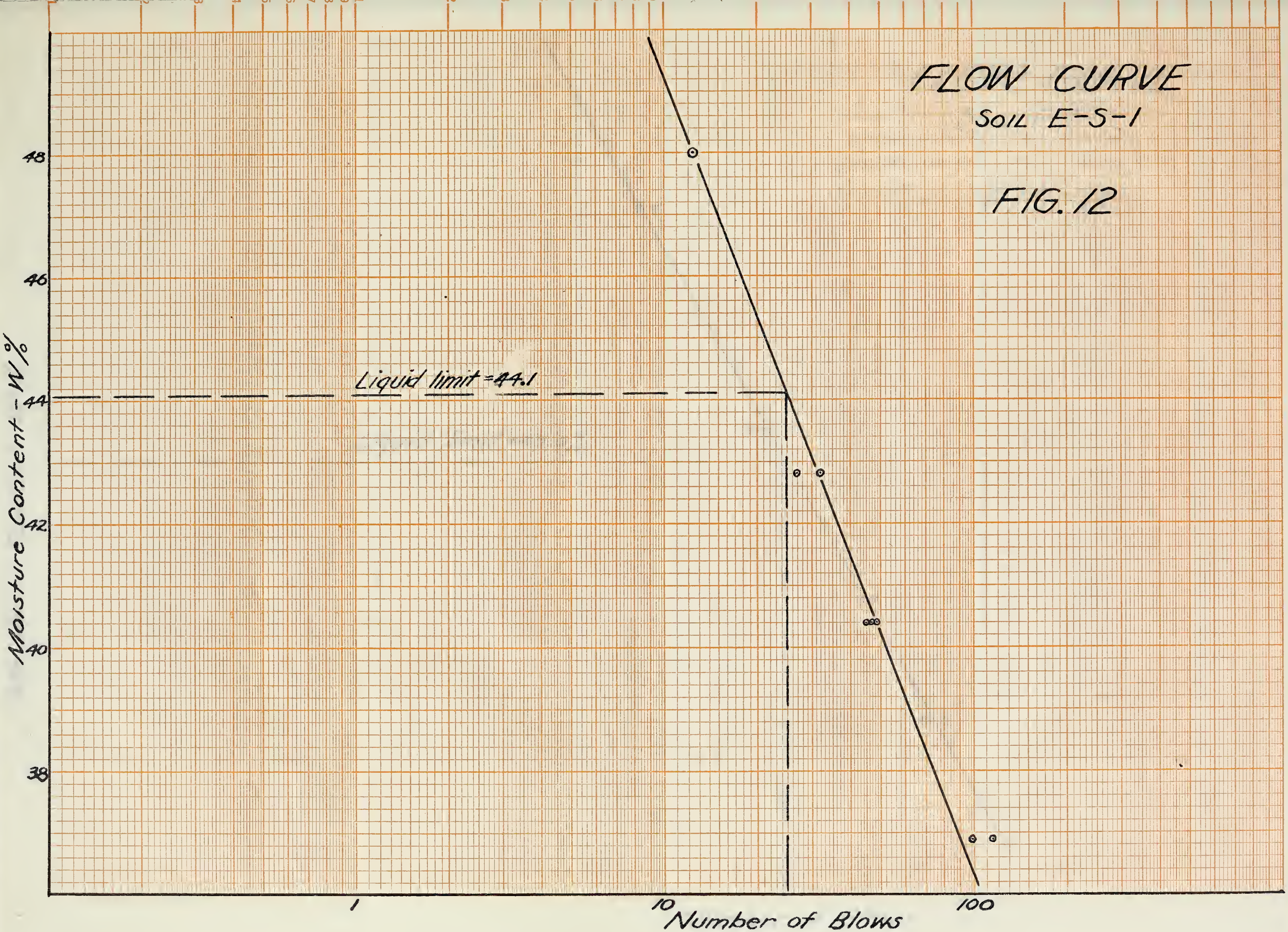
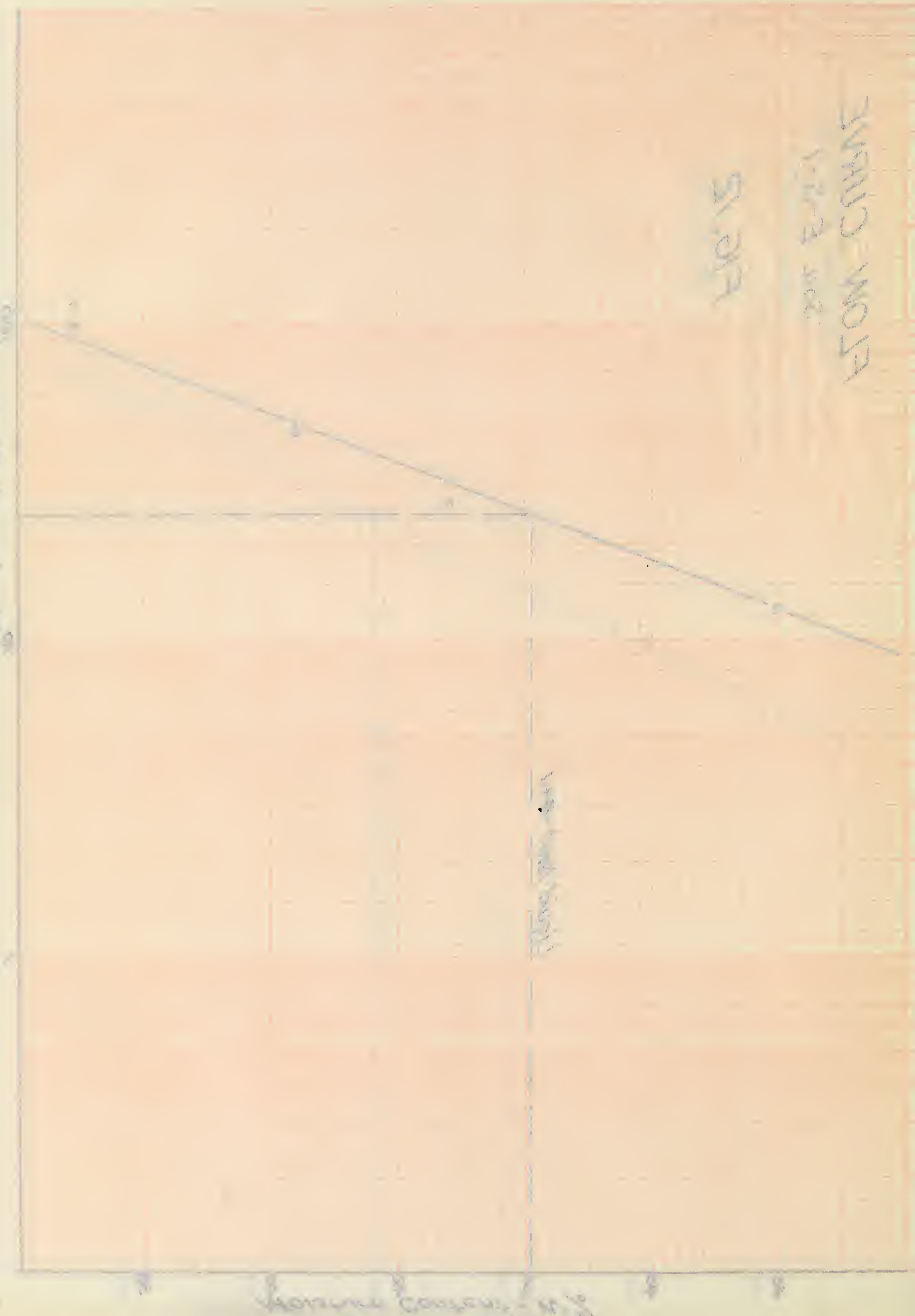

FLOW CURVE
SOIL E-S-1
Liquid limit = 44.1
48
46
44
42
40
38
Moisture Content - W%
1
10
100
Number of Blows

FIG. 12



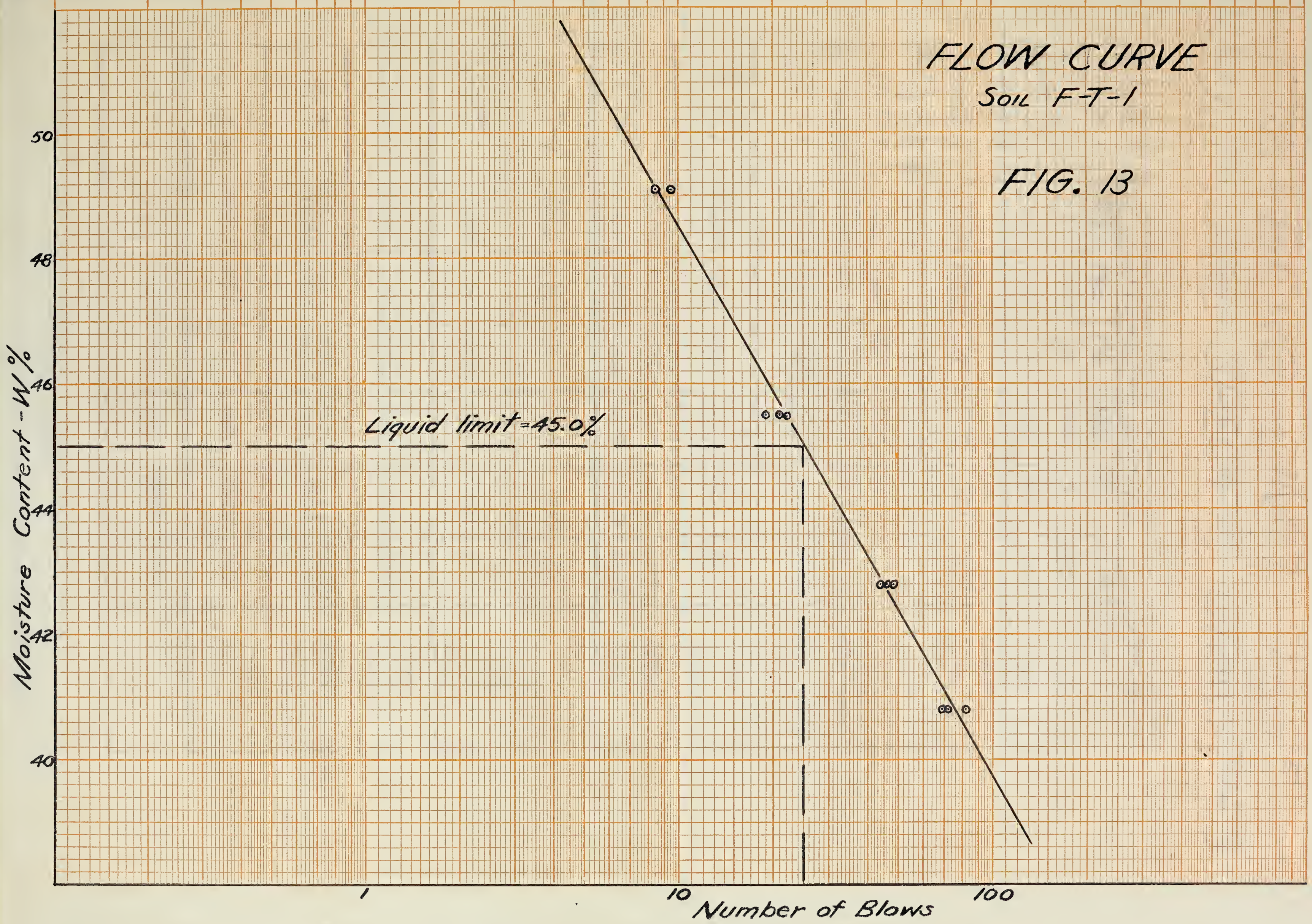
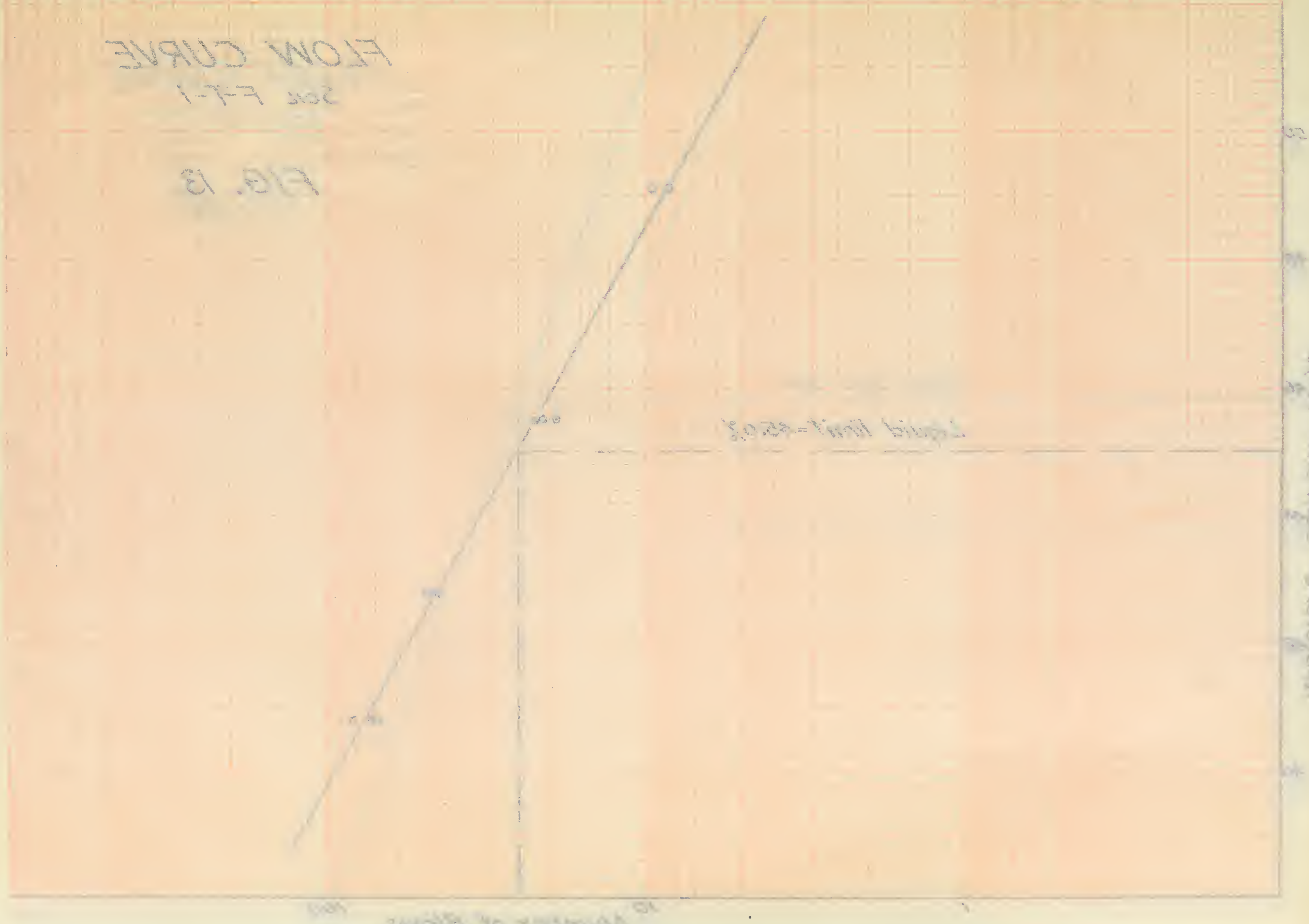

FLOW CURVE
SOIL F-T-1
Liquid limit = 45.0%
Moisture Content - W %
50
48
46
44
42
40
1
10
100
Number of Blows

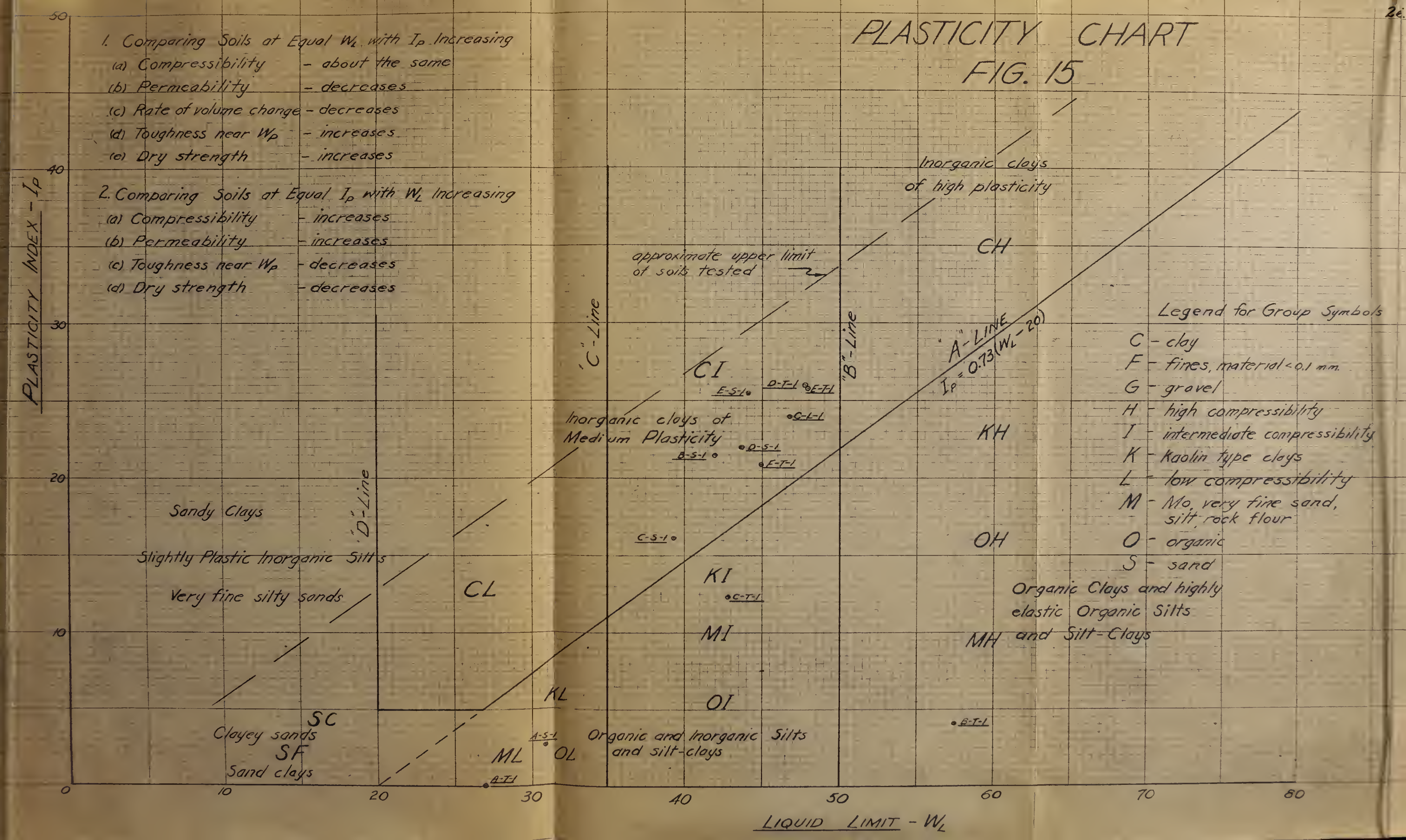
FIG. 13


FLOW CURVE
Soil F-7-1
FIG. 13
Liquid limit=45.0%
Number of Blows
Moisture Content - %

ATTERBERG LIMIT TEST RESULTS AND SPECIFIC GRAVITIES

SOIL NUMBER	ATTERBERG LIMITS						SPECIFIC GRAVITY
	W_L	W_P	W_I	W_S	F.I.	S.R.	
A-T-1	27.0 27.6	27.0 27.6	0 0	19.1 19.7	5.6 5.2	1.65 1.52	2.65 2.63
A-S-1	[illegible].9	28.3	2.6	[illegible]	5.5	1.74	2.64 2.65
B-T-1	[illegible].5	53.4	4.1	31.5	[illegible]	1.22	2.42 [illegible]
B-S-1	4[illegible]	20.5	21.5	15.0	[illegible].1	1.82	2.58 [illegible]
C-T-1	42.8	30.5	12.3	16.4	4.3	1.65	2.60 2.61
C-L-1	46.8 44.4	22.8 26.2	24.0 18.2	[illegible] 12.8	10.8 12.9	1.96 1.83	2.66 2.65
C-S-1	39.3	23.2	16.1	14.3	14.0	1.82	[illegible] 2.59
D-T-1	47.8	21.8	26.0	13.7	15.6	1.87	2.56 2.54
D-S-1	43.[illegible]	[illegible]1.6	22.0	15.6	13.3	1.84	2.66 2.64
E-T-1	48.0	22.1	25.9	12.7	10.3	1.85	2.60 2.65
E-S-1	44.1	18.6	25.5	14.3	12.9	1.89	2.67 2.70
F-T-1	45.0	[illegible]4.3	20.7	10.6	8.8	1.76	[illegible] [illegible]

FIG. 14



PLASTICITY CHART

FIG. 15

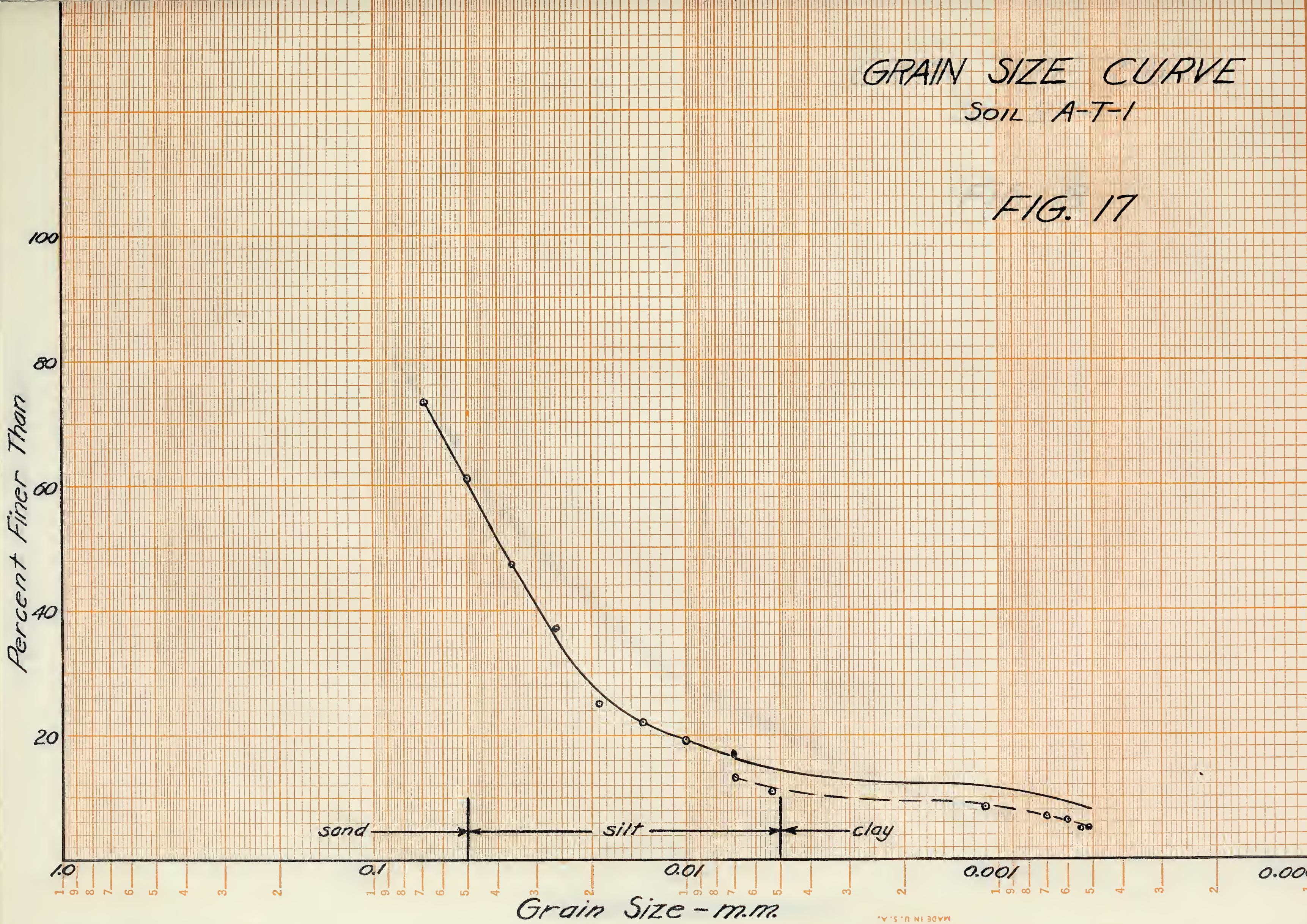
U.S.E.D. AND CASAGRANDE CLASSIFICATION

1		2	3	4		5	6	7	7a	8	9	
MAJOR DIVISIONS		SOIL GROUPS AND TYPICAL NAMES	SUGGESTED GROUP SYMBOLS	GENERAL IDENTIFICATION		OBSERVATIONS AND TESTS RELATING TO MATERIAL IN PLACE	PRINCIPAL CLASSIFICATION TESTS ON UNDISTURBED SAMPLES	VALUE AS FOUNDATION WHEN NOT SUBJECT TO FROST ACTION	VALUE AS WEARING SURFACE WITH SATISFACTORY DUST PALLIATIVE	VALUE AS BASE DIRECTLY UNDER WEARING SURFACE	POTENTIAL FROST ACTION	SHRINKAGE EXPANSION ELASTICITY
				Dry Strength	Other Pertinent Exams							
COARSE GRADED SOILS	GRAVEL AND GRAVELLY SOILS	Well Graded Gravel or Gravel-Sand Mixtures Little or No Fines	G.W.	NONE	Gradation, Grain Shape	Dry Unit Weight or Void Ratio	Mechanical Analysis	Excellent	Poor	Good To Excellent	None To Very Slight	Almost None
		Well Graded Gravel-Sand-Clay Mixtures, Excellent Binder	G.C.	Medium To High	Gradation, Grain Shape, Binder Exam Wet and Dry	Degree of Compaction	Mechanical Analysis, Liquid & Plastic Limits on Binder	Excellent	Excellent	Fair To Excellent	Medium	Very Slight
		Poorly Graded Gravel & Gravel-Sand Mixtures Little or No Fines	G.P	Very Slight To High	Gradation, Grain Shape, Binder Exam Wet and Dry	Cementation Durability of Grains	Mechanical Analysis	Good To Excellent	Poor	Poor To Good	None To Very Slight	Almost None
		Gravel With Fines, Very Silty Gravel, Clayey Gravel, Poorly Graded Gravel-Sand Clay Mixtures	G.F.	NONE	Gradation Grain Shape	Stratifications & Drainage Characteristics	Mech. Analysis. Liquid & Plastic Limits on Binder if Applicable	Good To Excellent	Fair To Good	Poor To Good	Slight To Medium	Almost None To Slight
	SANDS AND SANDY SOILS	Well Graded Sands and Gravelly Sands, Little or No Fines	S.W.	NONE	Gradation Grain Shape	Ground Water Conditions	Mechanical Analysis	Good To Excellent	Poor	Poor To Good	None To Very Slight	Almost None
		Well Graded Sand-Clay Mixtures, Excellent Binder	S.C.	Medium To High	Gradation, Grain Shape Binder Exam Wet and Dry	Traffic Tests	Mechanical Analysis Liquid and Plastic Limits on Binder	Good To Excellent	Excellent	Poor To Good	Medium	Very Slight
		Poorly Graded Sands Little or No Fines	S.P.	NONE	Gradation, Grain Shape	Large Scale Tests Or	Mechanical Analysis	Fair To Good	Poor	Not Suitable	None To Very Slight	Almost None
		Sand With Fines, Very Silty Sands, Clayey Sands, Poorly Graded Sand-Clay Mixtures	S.F.	Very Slight To High	Gradation Grain Shape Binder Exam Wet and Dry	California Bearing Tests	Mech. Analysis Liquid & Plastic Limits on Binder if Applicable	Fair To Good	Fair To Good	Not Suitable	Slight To High	Almost None To Medium
FINE GRAINED SOILS CONTAINING LITTLE OR NO COARSE GRAINED MATERIALS	FINE GRAINED SOILS HAVING LOW TO MEDIUM COMPRESSIBILITY	Silts (Inorganic) & Very Fine Sands MO, Rock Flour, Silty or Clayey Fine Sands With Slight Plasticity	M.L.	Very Slight To Medium	Examination Wet (Shaking Test & Plasticity)	Dry Unit Weight, Water Content & Void Ratio	Mech. Analysis. Liquid & Plastic Limits on Binder if Applicable	Fair To Poor	Poor	Not Suitable	Medium To Very High	Slight To Medium
		Clays (Inorganic) of Low To Medium Plasticity Sandy Clays, Silty Clays Lean Clays	C.L.	Medium To High	Examination In The Plastic Range.	Consistency Undisturbed & Remolded	Liquid & Plastic Limits	Fair To Poor	Poor	Not Suitable	Medium To High	Medium
		Organic Silts & Organic Silt-Clays of Low Plasticity	O.L.	Slight To Medium	Examination In The Plastic Range. Odor	Stratification, Root Holes, Fissures, etc	Liquid & Plastic from Natural Condition & After Oven Drying	Poor	Very Poor	Not Suitable	Medium To High	Medium To High
	FINE GRAINED SOILS HAVING HIGH COMPRESSIBILITY	Micaceous or Diatomaceous Fine Sandy & Silty Soils, Elastic Silts	M.H.	Very Slight To Medium	Examination Wet (Shaking Test & Plasticity)	Drainage & Ground Water Conditions	Mech. Analysis. Liquid & Plastic Limits on Binder if Applicable	Poor	Very Poor	Not Suitable	Medium To Very High	High
		Clays (Inorganic) of High Plasticity, Fat Clays	C.H	High	Examination In The Plastic Range	Traffic Tests, Large Scale Load Tests	Liquid & Plastic Limits	Poor To Very Poor	Very Poor	Not Suitable	Medium	High
		Organic Clays of Medium To High Plasticity	O.H.	High	Examination In The Plastic Range. Odor.	California Bearing Tests, or Compression Tests	Liquid & Plastic Lts. from Natural Conditions & After Oven Drying	Very Poor	Useless	Not Suitable	Medium	High
FIBROUS ORGANIC SOILS WITH VERY HIGH COMPRESSIBILITY		Peat & Other Highly Organic Swamp Soils	Pt.	Readily Identified		Consistency, Texture & Natural Water Content		Extremely Poor	Useless	Not Suitable	Slight	Very High

FIG. 16

DE CLASSIFICATION

	7a	8	9	10	11	12	13	14	15
I- N	VALUE AS WEARING SURFACE WITH SATISFACTORY DUST PALLIATIVE	VALUE AS BASE DIRECTLY UNDER WEARING SURFACE	POTENTIAL FROST ACTION	SHRINKAGE EXPANSION ELASTICITY	DRAINAGE CHARACTERISTICS	COMPACTION CHARACTERISTICS AND EQUIPMENT	SOLIDS AT OPT. COMPACTION lb/cu ft & VOID RATIO	CALIF. BRG. RATIO FOR COMPACTED & SOAKED SPECIMEN	COMPARABLE GROUPS IN PUB. ROADS CLASSIFICATION
	Poor	Good To Excellent	None To Very Slight	Almost None	Excellent	Excellent, Tractor	>125 e<0.35	>50	A-3
	Excellent	Fair To Excellent	Medium	Very Slight	Practically Impervious	Excellent, Tamping Roller	>130 e<0.30	>40	A-1
	Poor	Poor To Good	None To Very Slight	Almost None	Excellent	Good, Tractor	>115 e<0.45	25-60	A-3
	Fair To Good	Poor To Good	Slight To Medium	Almost None To Slight	Fair To Practically Impervious	Good, Close Control Essential, Rubber-Tired Roller Tractor	>120 e<0.40	>20	A-2
	Poor	Poor To Good	None To Very Slight	Almost None	Excellent	Excellent, Tractor	>120 e<0.40	20-60	A-[illegible]
	Excellent	Poor To Good	Medium	Very Slight	Practically Impervious	Excellent, Tamping Roller	>125 e<0.35	20-60	A-1
	Poor	Not Suitable	None To Very Slight	Almost None	Excellent	Good, Tractor	>100 e<0.70	10-30	A-3
	Fair To Good	Not Suitable	Slight To High	Almost None To Medium	Fair To Practically Impervious	Good, Close Control Essential, Rubber-Tired Roller	>105 e<0.60	8-30	A-2
	Poor	Not Suitable	Medium To Very High	Slight To Medium	Fair To Poor	Good To Poor, Close Control Essential Rubber-Tired Roller	>100 e<0.70	6-25	A-4 A-6 A-7
	Poor	Not Suitable	Medium To High	Medium	Practically Impervious	Fair To Good Tamping Roller	>100 e<0.70	4-15	A-4 A-6 A-7
	Very Poor	Not Suitable	Medium To High	Medium To High	Poor	Fair To Poor Tamping Roller	>90 e<0.90	3-8	A-4 A-7
	Very Poor	Not Suitable	Medium To Very High	High	Fair To Poor	Poor To Very Poor	<100 e>0.70	<7	A-5
	Very Poor	Not Suitable	Medium	High	Practically Impervious	Fair To Poor Tamping Roller	>90 e<0.90	<6	A-6 A-7
	Useless	Not Suitable	Medium	High	Practically Impervious	Poor To Very Poor	<100 e>0.70	<4	A-7 A-8
or	Useless	Not Suitable	Slight	Very High	Poor To Very Poor	Compaction Not Practical			A-8


GRAIN SIZE CURVE
SOIL A-T-1
FIG. 17
Percent Finer Than
100
80
60
40
20
sand
silt
clay
1.0
0.1
0.01
0.001
0.0001
Grain Size - m.m.

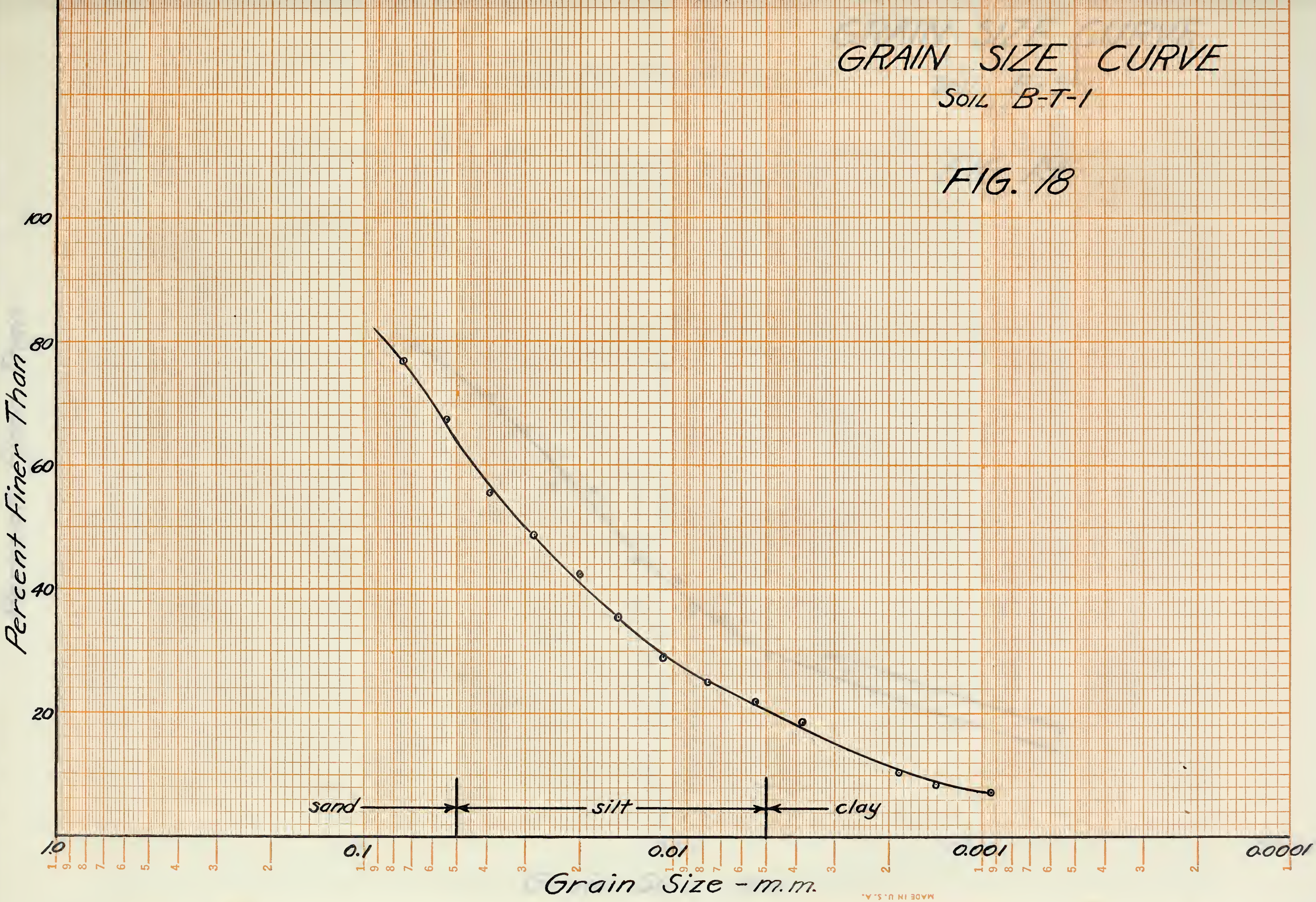
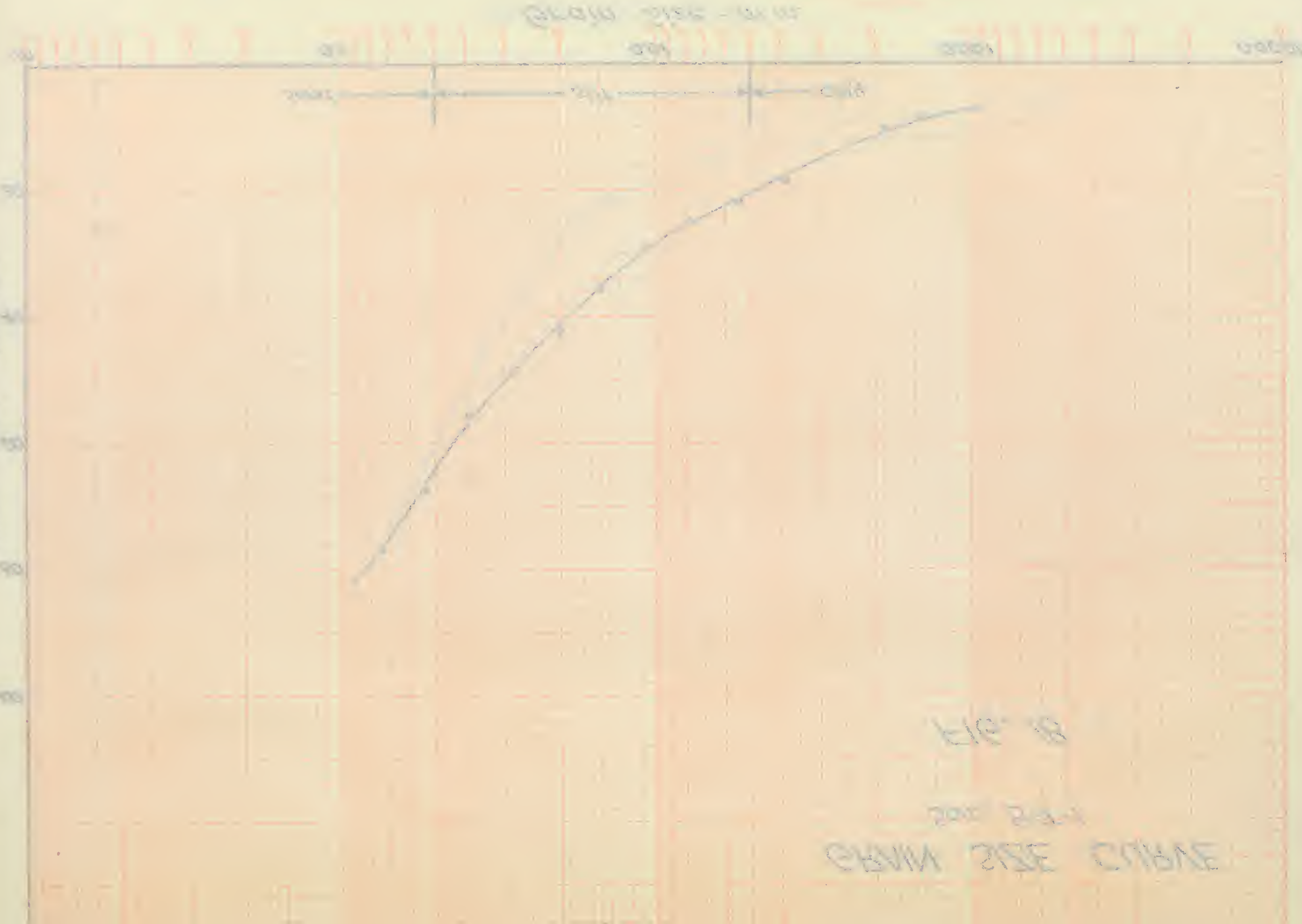
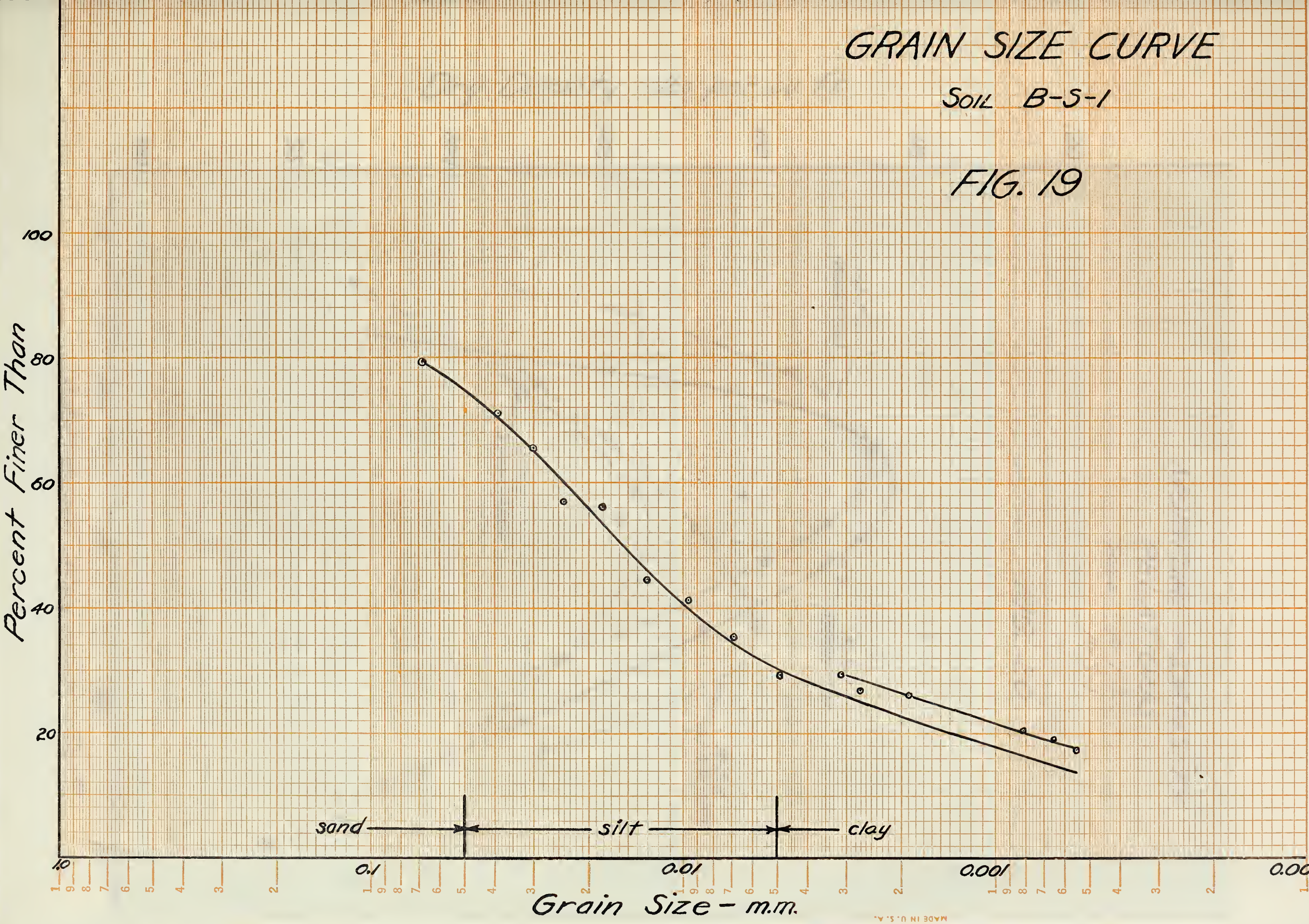

GRAIN SIZE CURVE
SOIL B-T-1
100
80
60
40
20
Percent Finer Than
sand
silt
clay
1.0
0.1
0.01
0.001
0.0001
Grain Size - m.m.

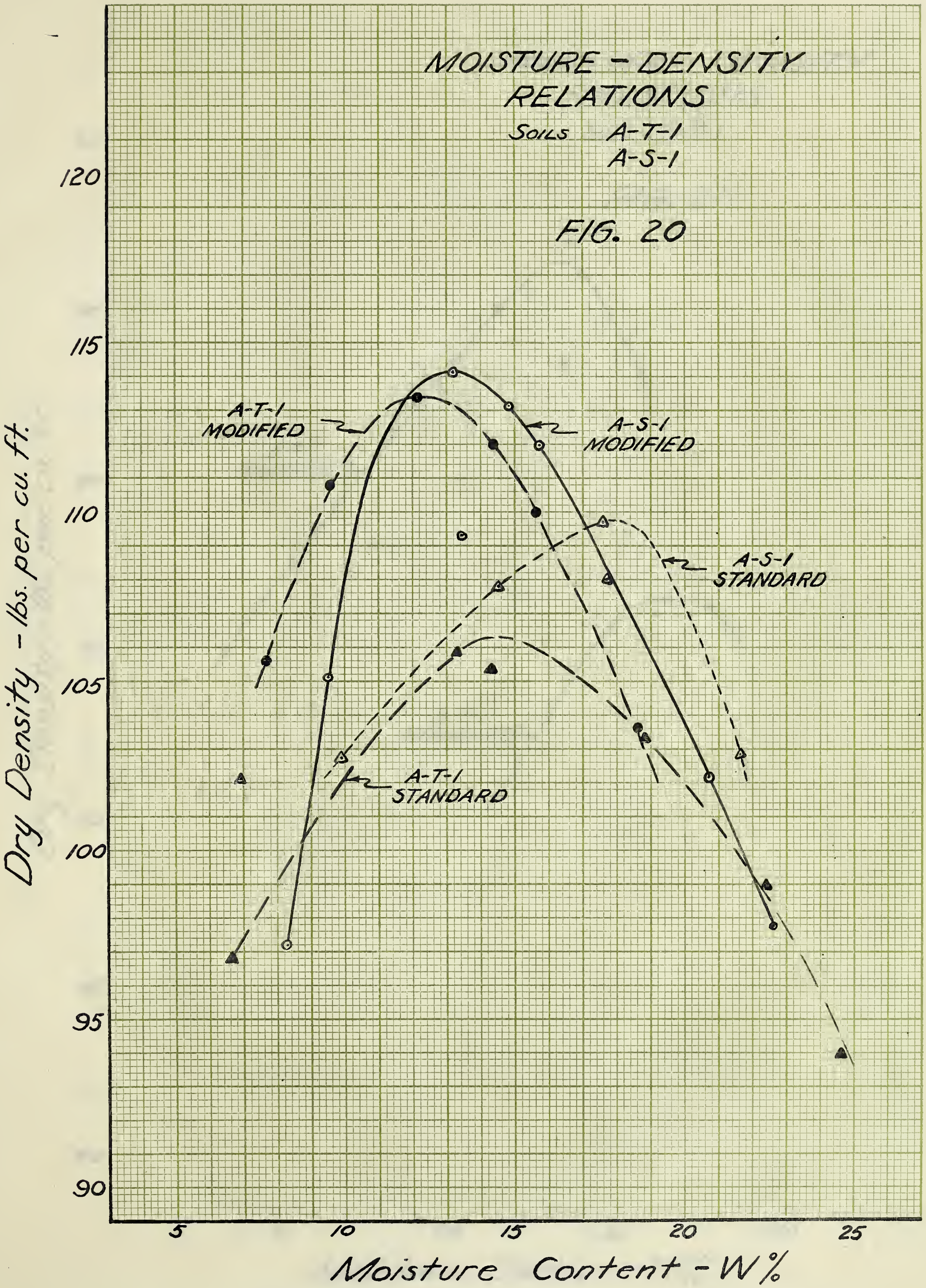
FIG. 18

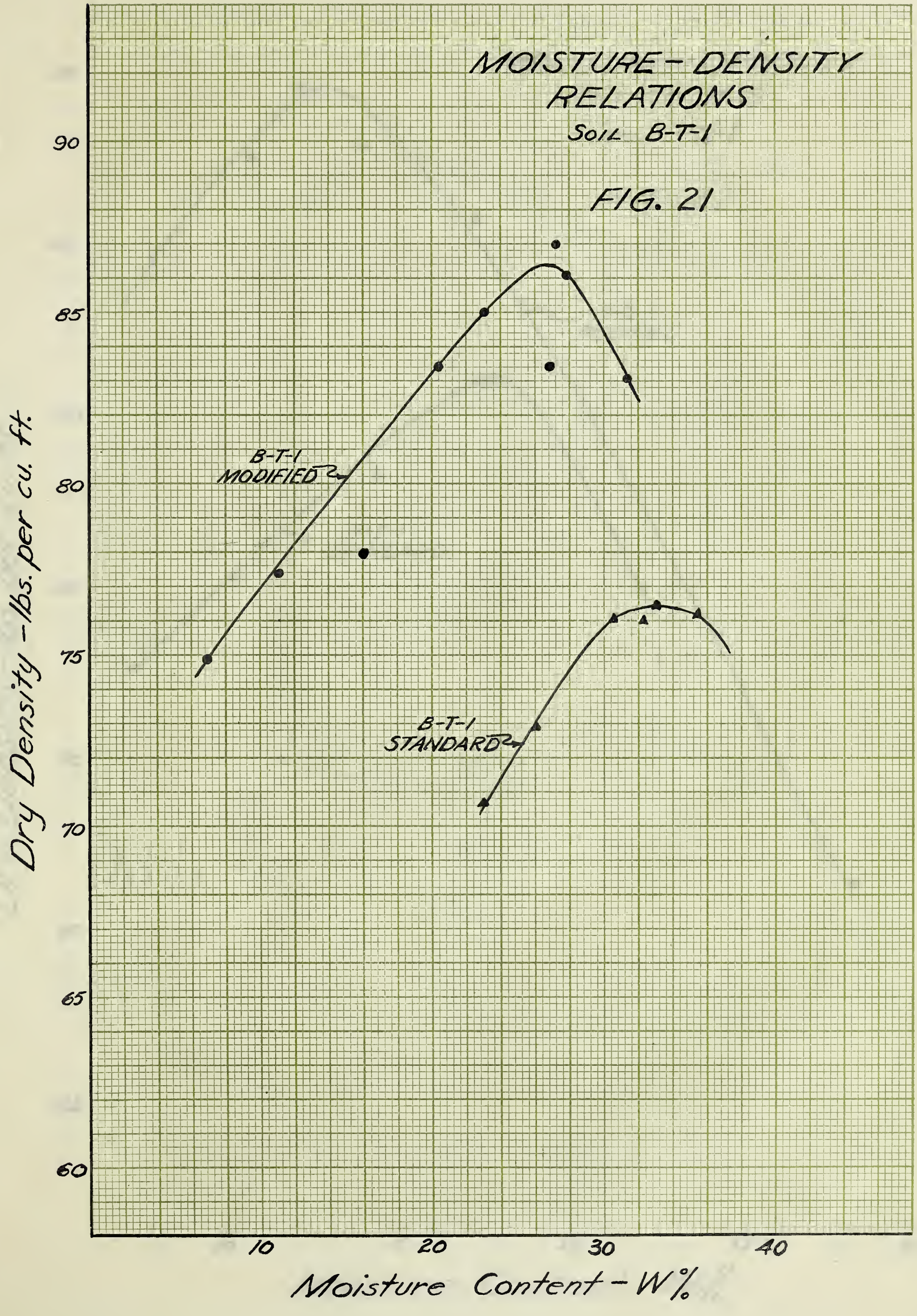


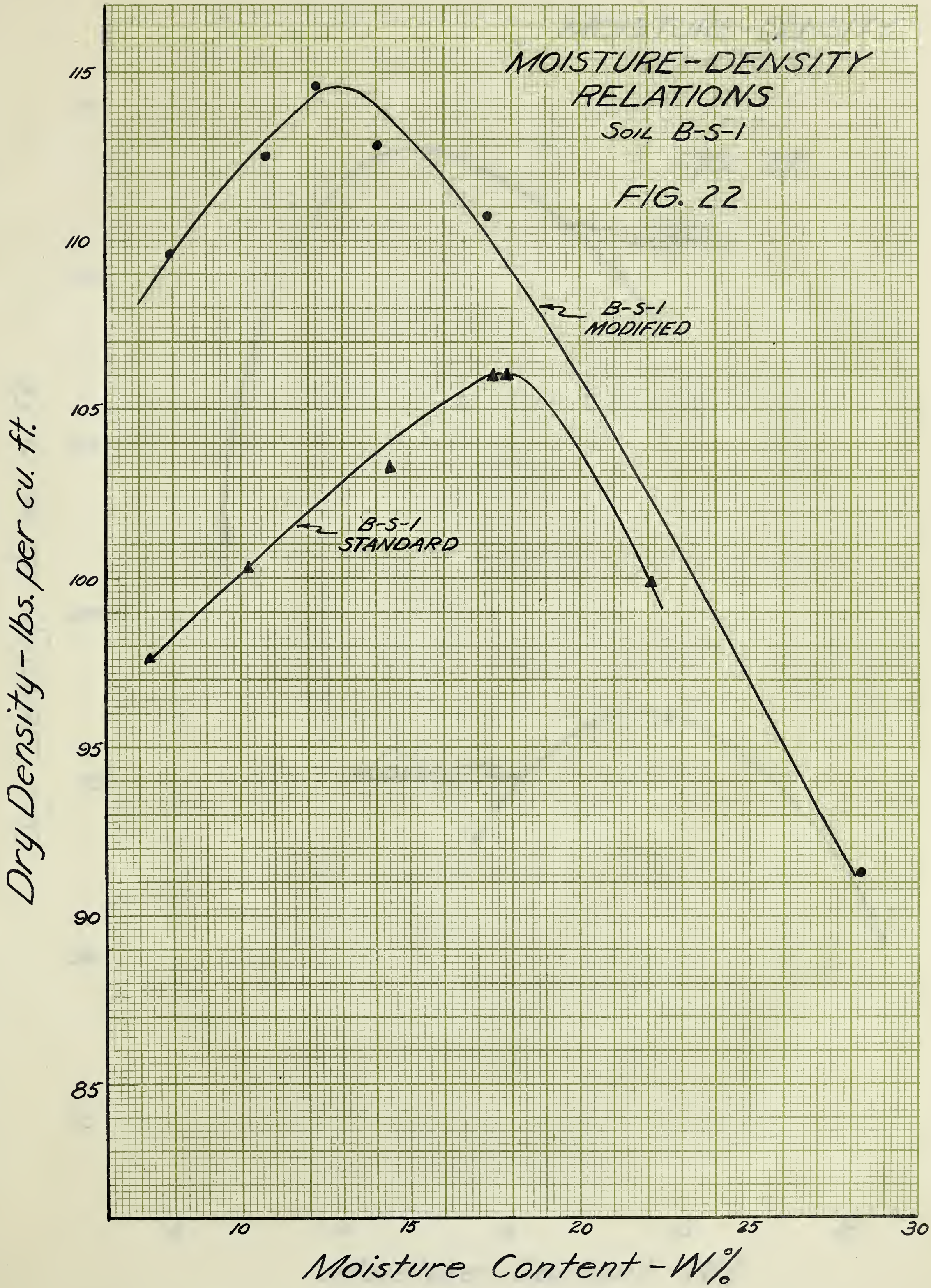


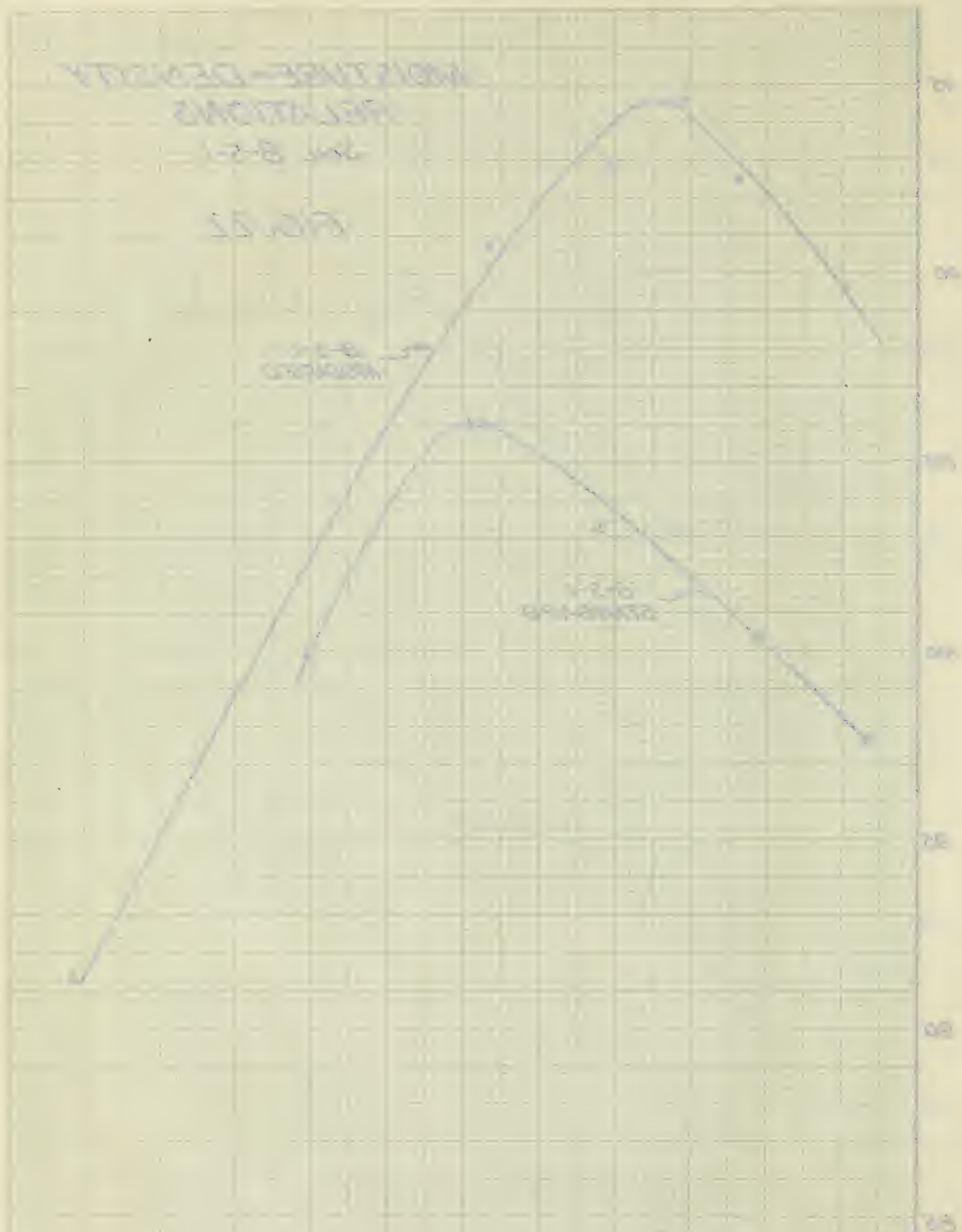
GRAIN SIZE CURVE
SOIL B-S-1
Percent Finer Than
100
80
60
40
20
sand
silt
clay
1.0
0.1
0.01
0.001
0.0001
Grain Size - m.m.

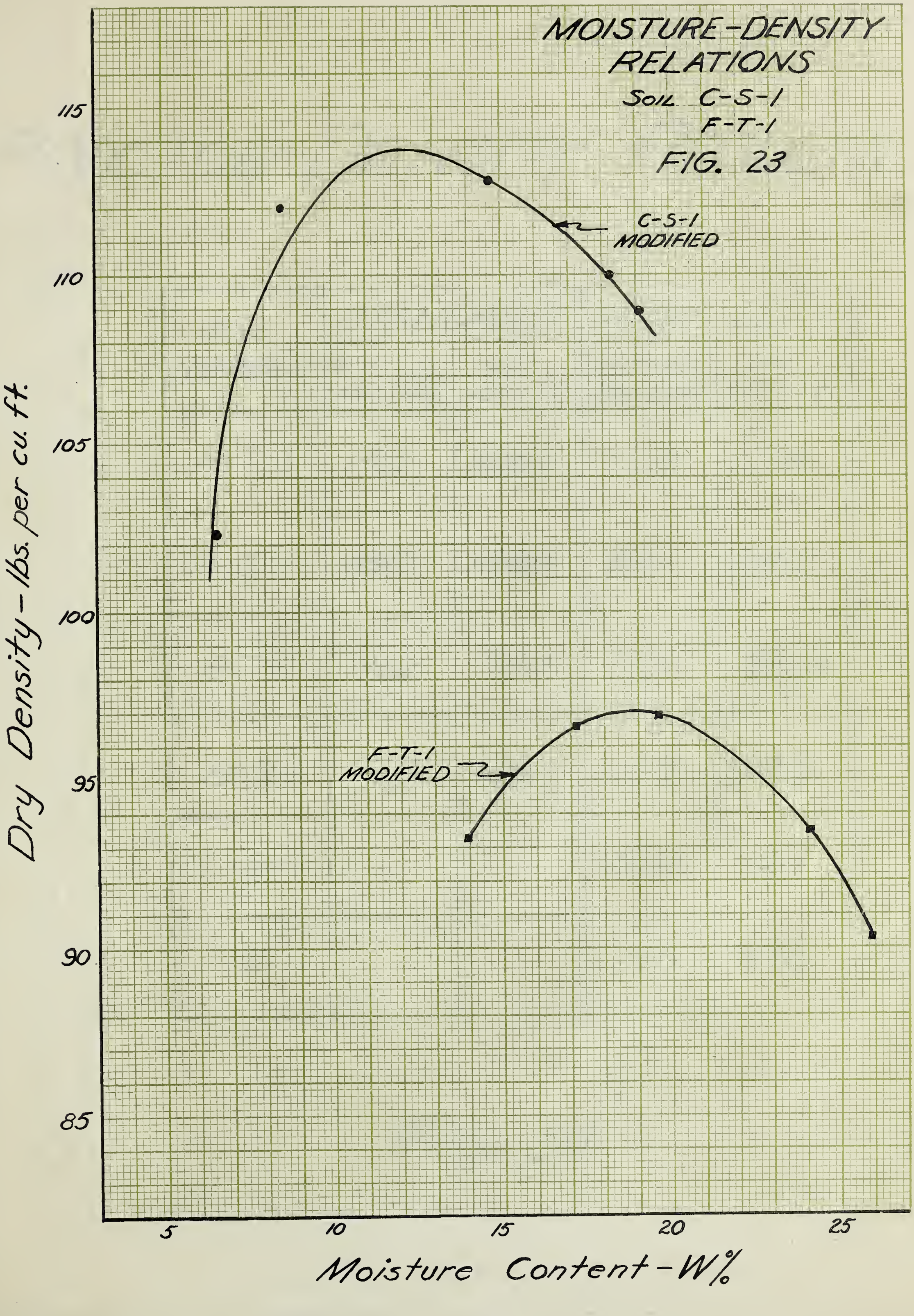
FIG. 19


MOISTURE - DENSITY
RELATIONS
Soils A-T-1
A-S-1
FIG. 20
120
115
110
105
100
95
90
Dry Density - lbs. per cu. ft.
A-T-1
MODIFIED
A-S-1
MODIFIED
A-S-1
STANDARD
A-T-1
STANDARD
5
10
15
20
25
Moisture Content - W%


MOISTURE - DENSITY RELATIONS
SOIL B-T-1
FIG. 21
B-T-1 MODIFIED
B-T-1 STANDARD
90
85
80
75
70
65
60
10
20
30
40
Dry Density - lbs. per cu. ft.
Moisture Content - W%


MOISTURE-DENSITY
RELATIONS
SOIL B-S-1
FIG. 22
B-S-1
MODIFIED
B-S-1
STANDARD
115
110
105
100
95
90
85
10
15
20
25
30
Dry Density - lbs. per cu. ft.
Moisture Content - W%

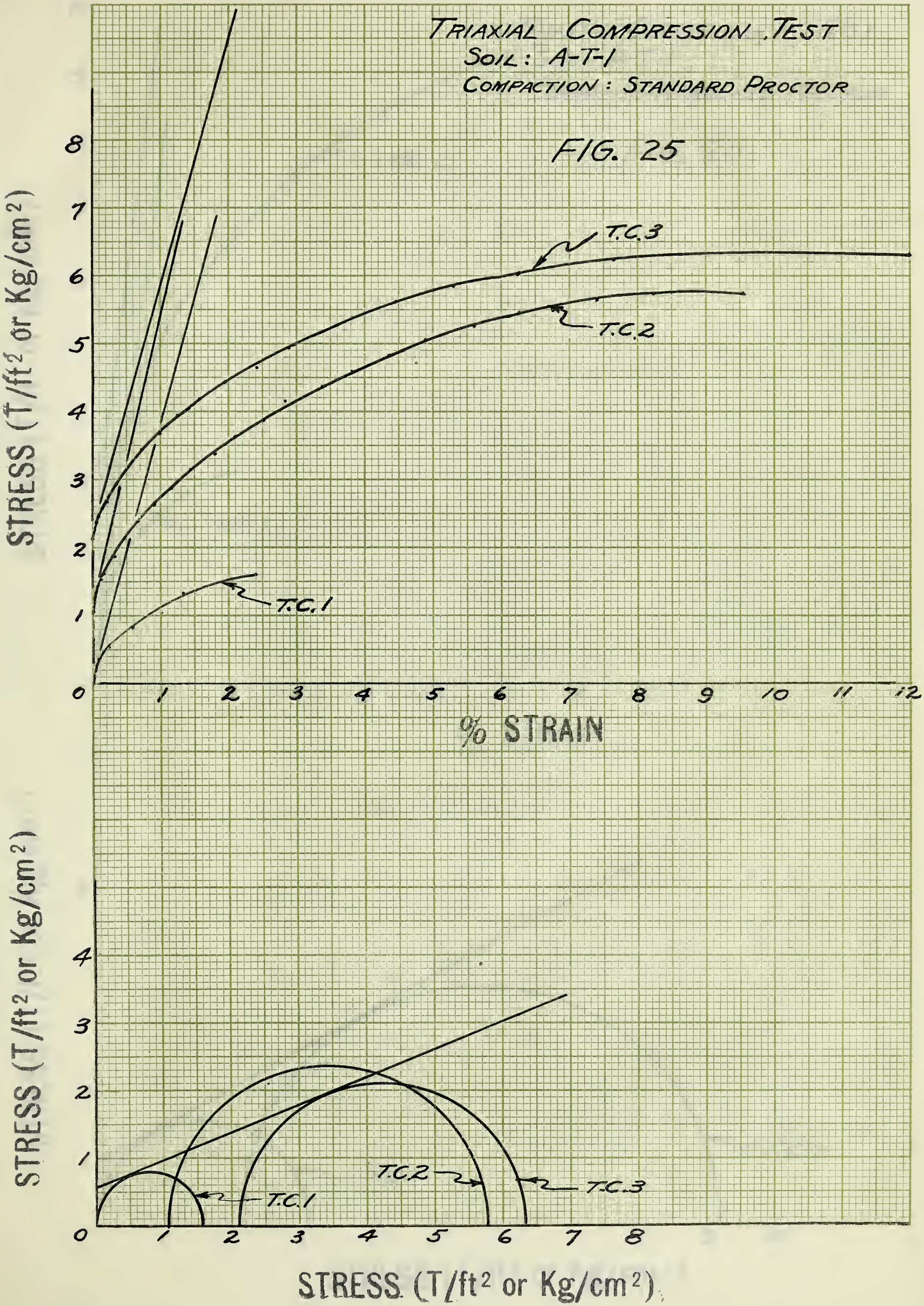


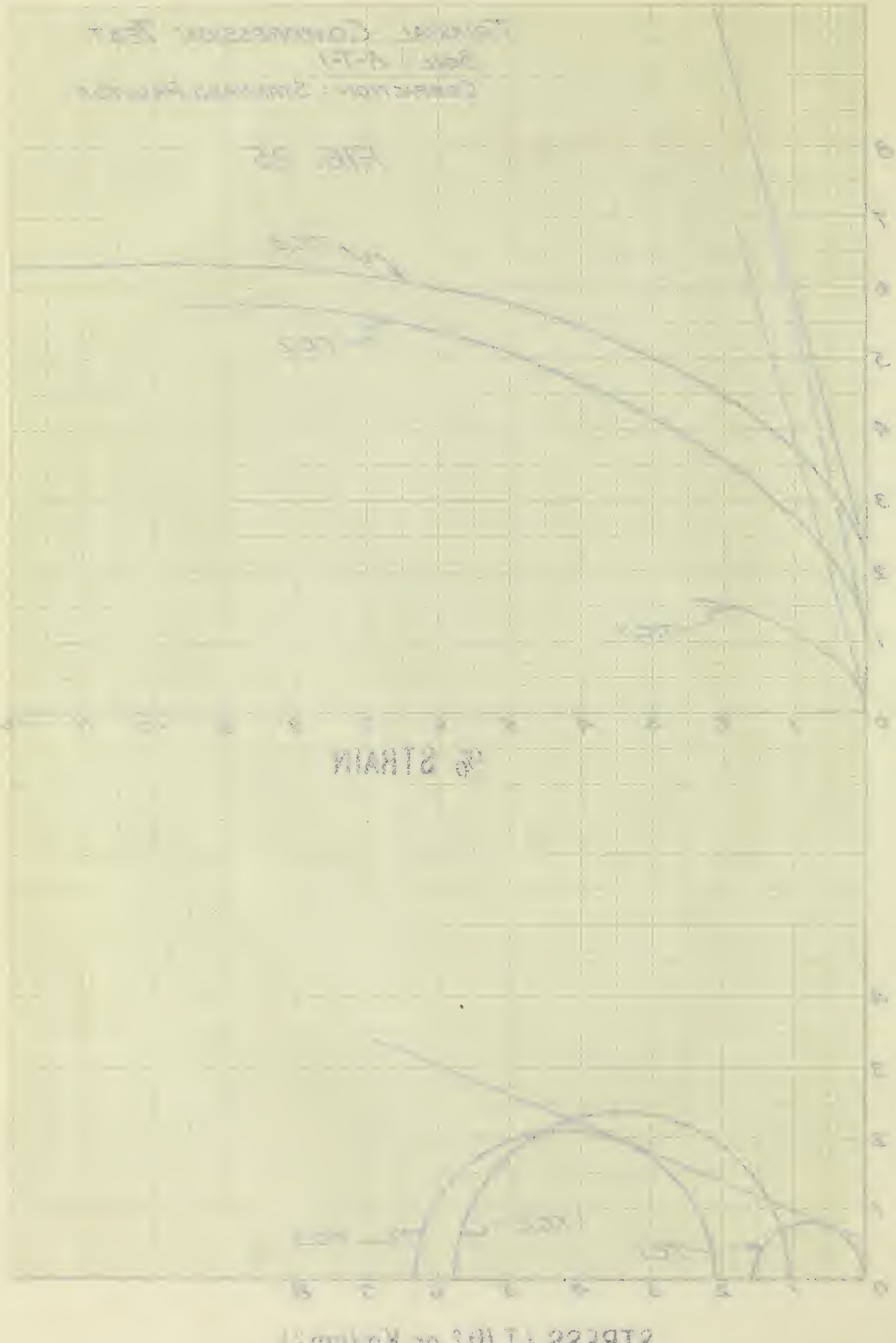

MOISTURE-DENSITY
RELATIONS
SOIL C-S-1
F-T-1
FIG. 23
C-S-1
MODIFIED
F-T-1
MODIFIED
115
110
105
100
95
90
85
5
10
15
20
25
Dry Density - lbs. per cu. ft.
Moisture Content - W%

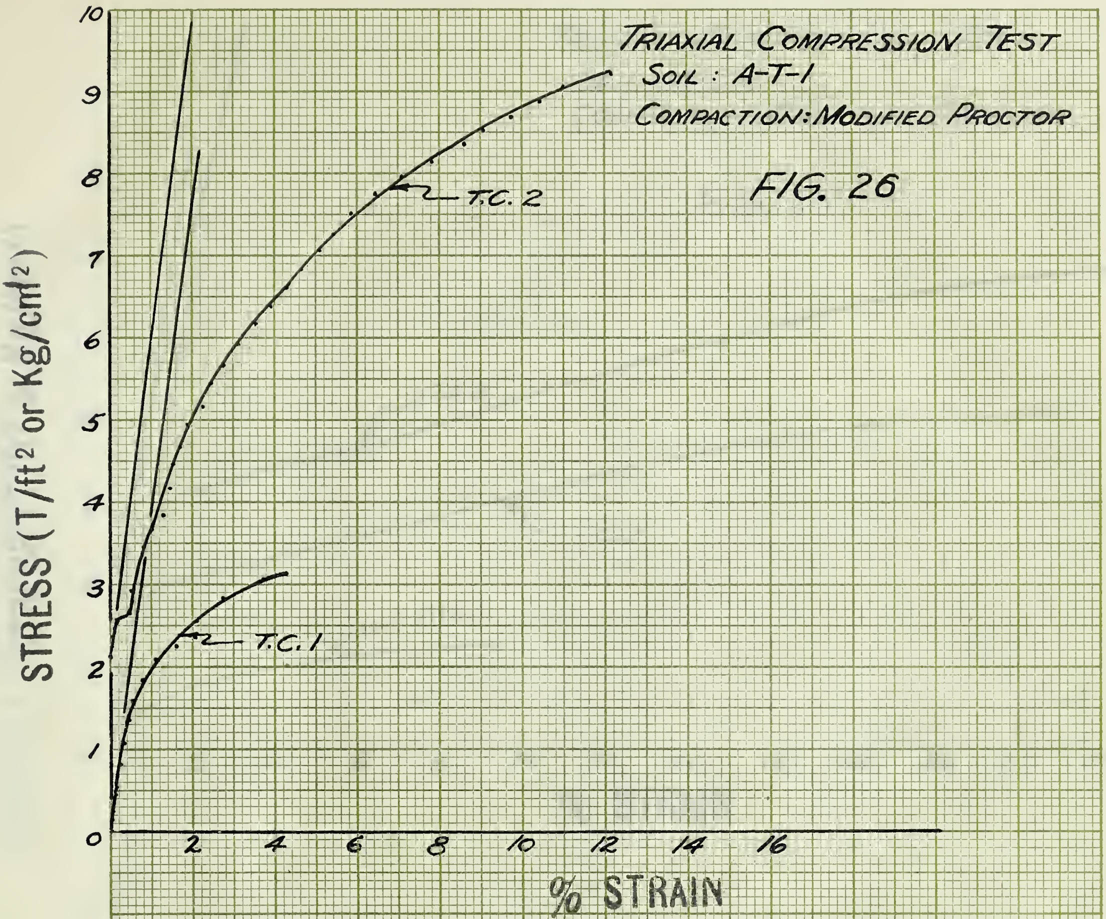
PROCTOR DENSITY RESULTS

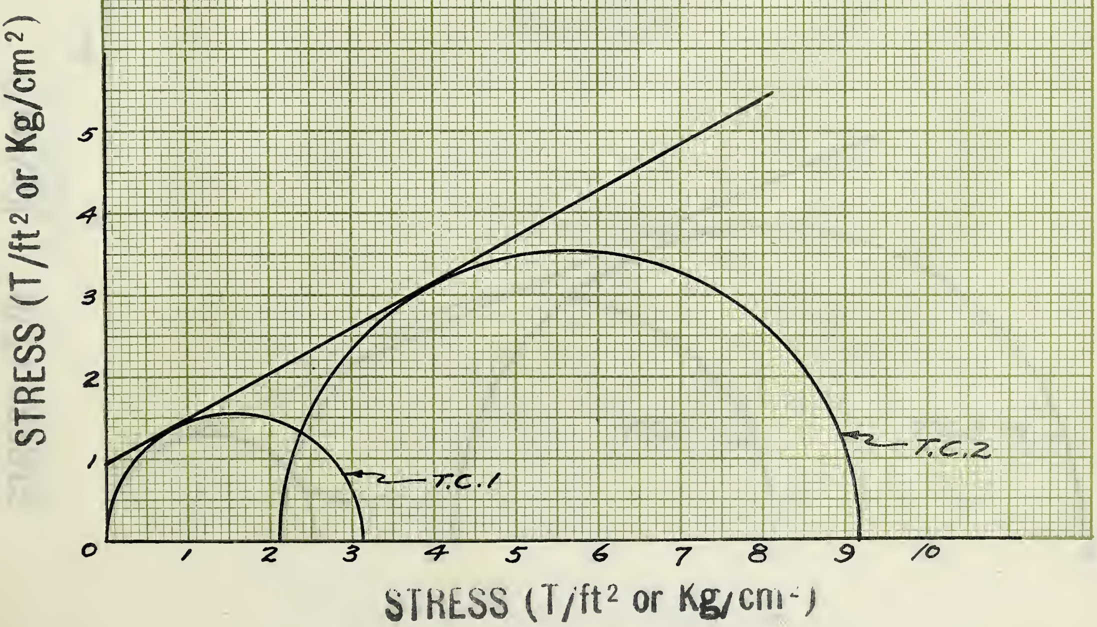
SOIL NUMBER	STANDARD		MODIFIED	
	OPTIMUM MOISTURE	DRY DENSITY	OPTIMUM MOISTURE	DRY DENSITY
A-T-1	14.6	106.3	12.4	113.4
A-S-1	18.0	109.8	13.1	114.2
B-T-1	33.2	76.4	26.5	86.4
B-S-1	18.0	106.1	12.7	114.6
C-S-1			12.0	113.7
F-T-1			19.0	97.0

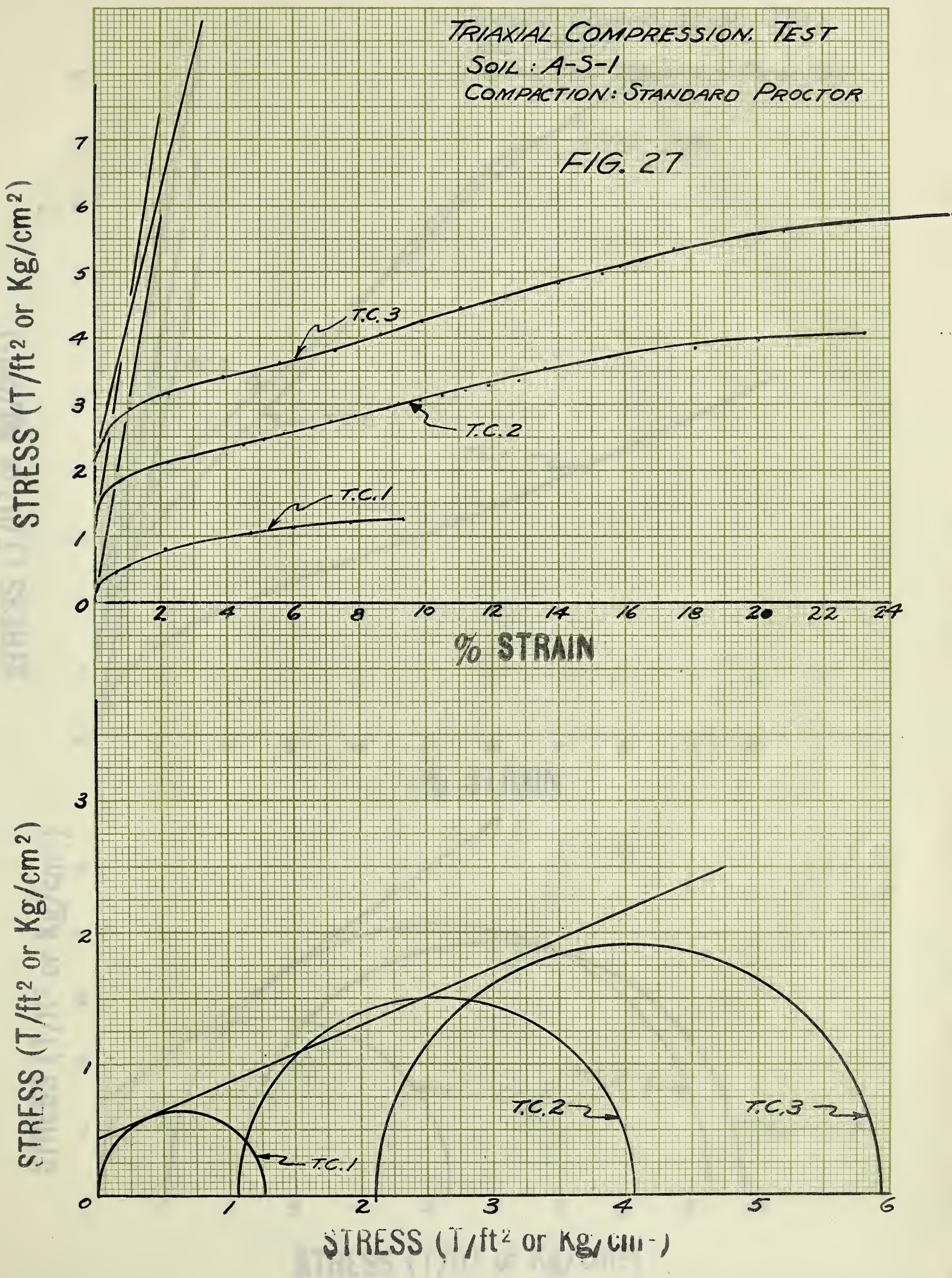
FIG. 24

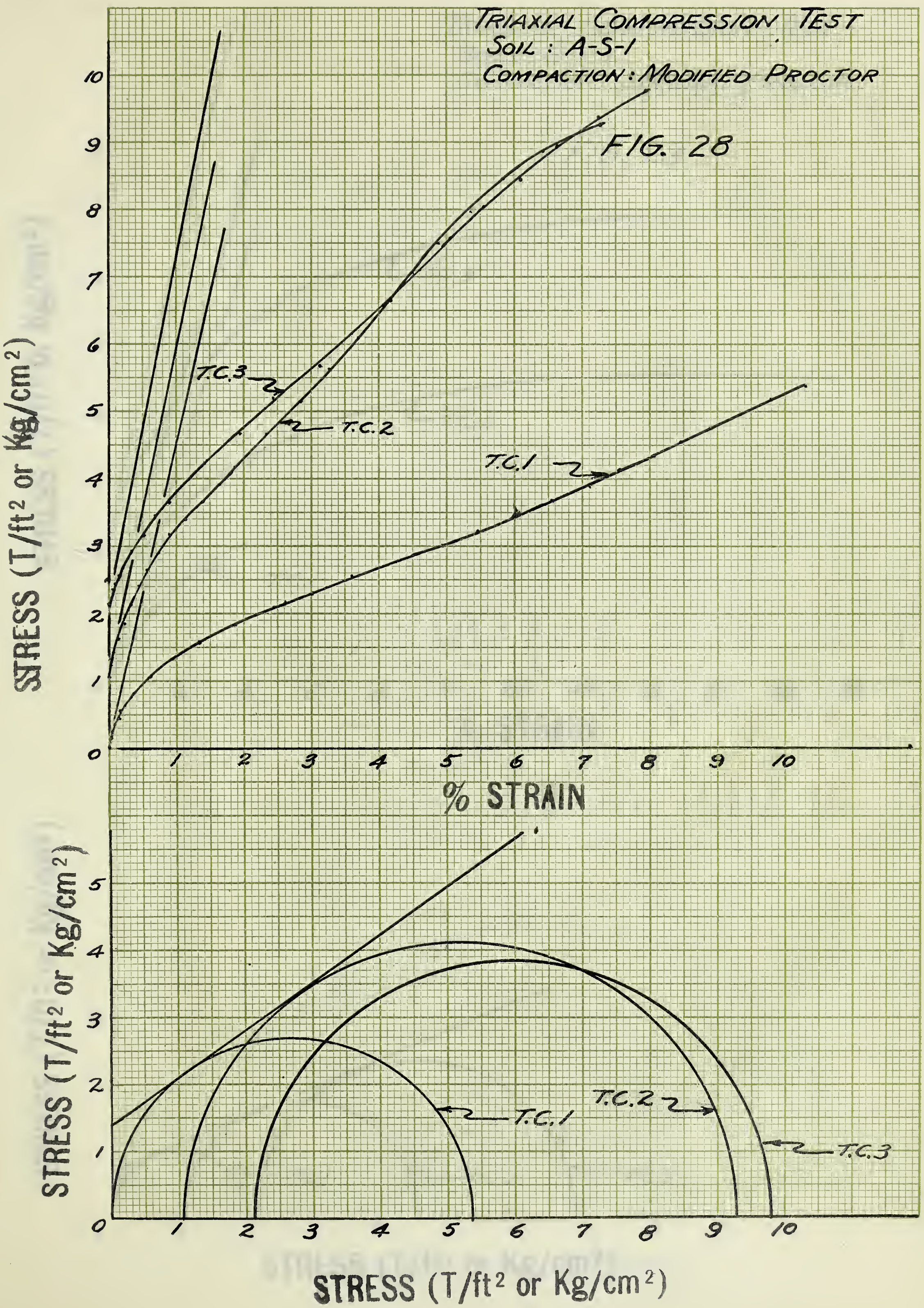

TRIAXIAL COMPRESSION TEST
SOIL: A-T-1
COMPACTION: STANDARD PROCTOR
FIG. 25
T.C.3
T.C.2
T.C.1
STRESS (T/ft² or Kg/cm²)
% STRAIN
0 1 2 3 4 5 6 7 8 9 10 11 12
1 2 3 4 5 6 7 8
T.C.2
T.C.3
T.C.1
STRESS (T/ft² or Kg/cm²)
0 1 2 3 4 5 6 7 8
1 2 3 4
STRESS (T/ft² or Kg/cm²)

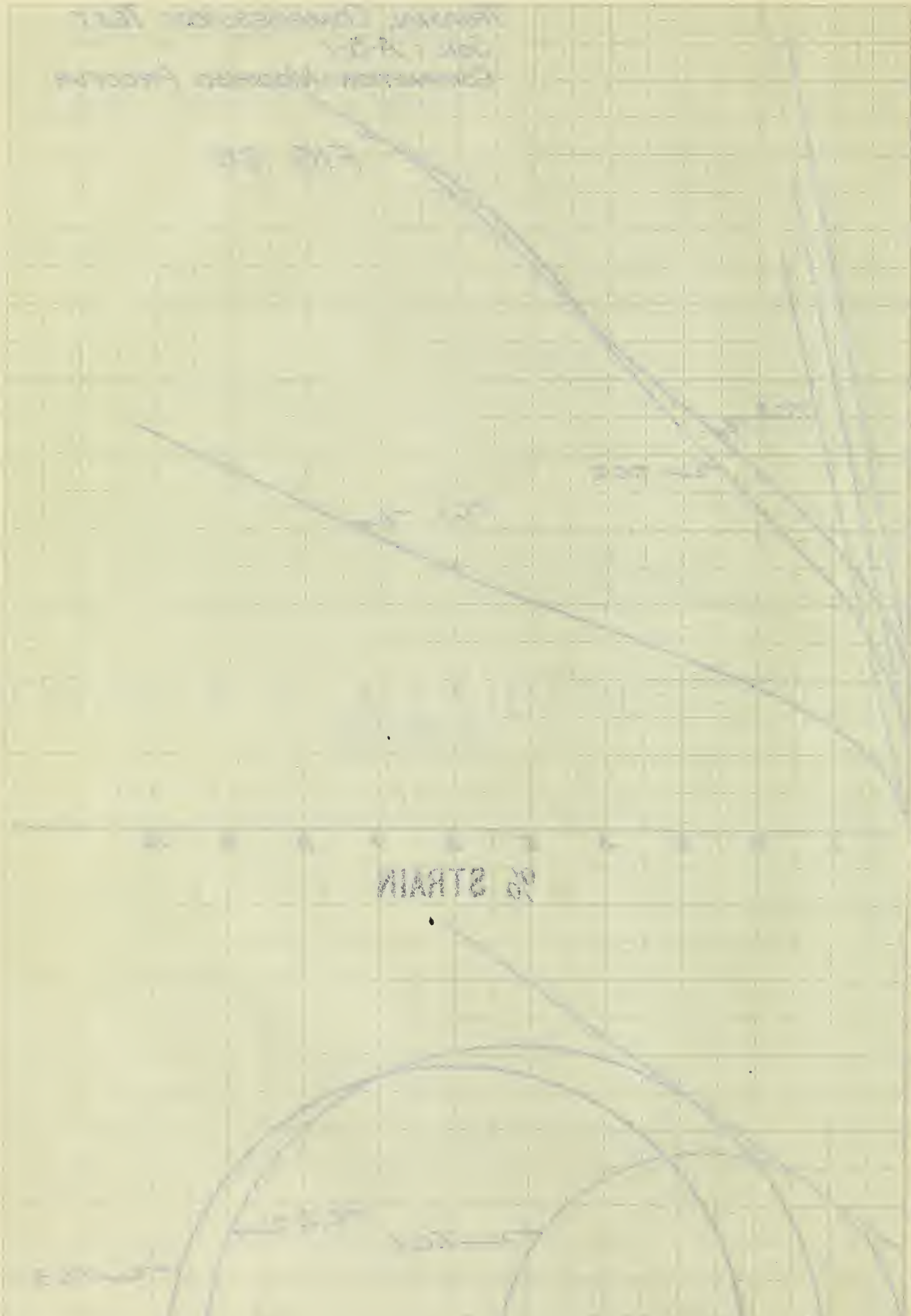


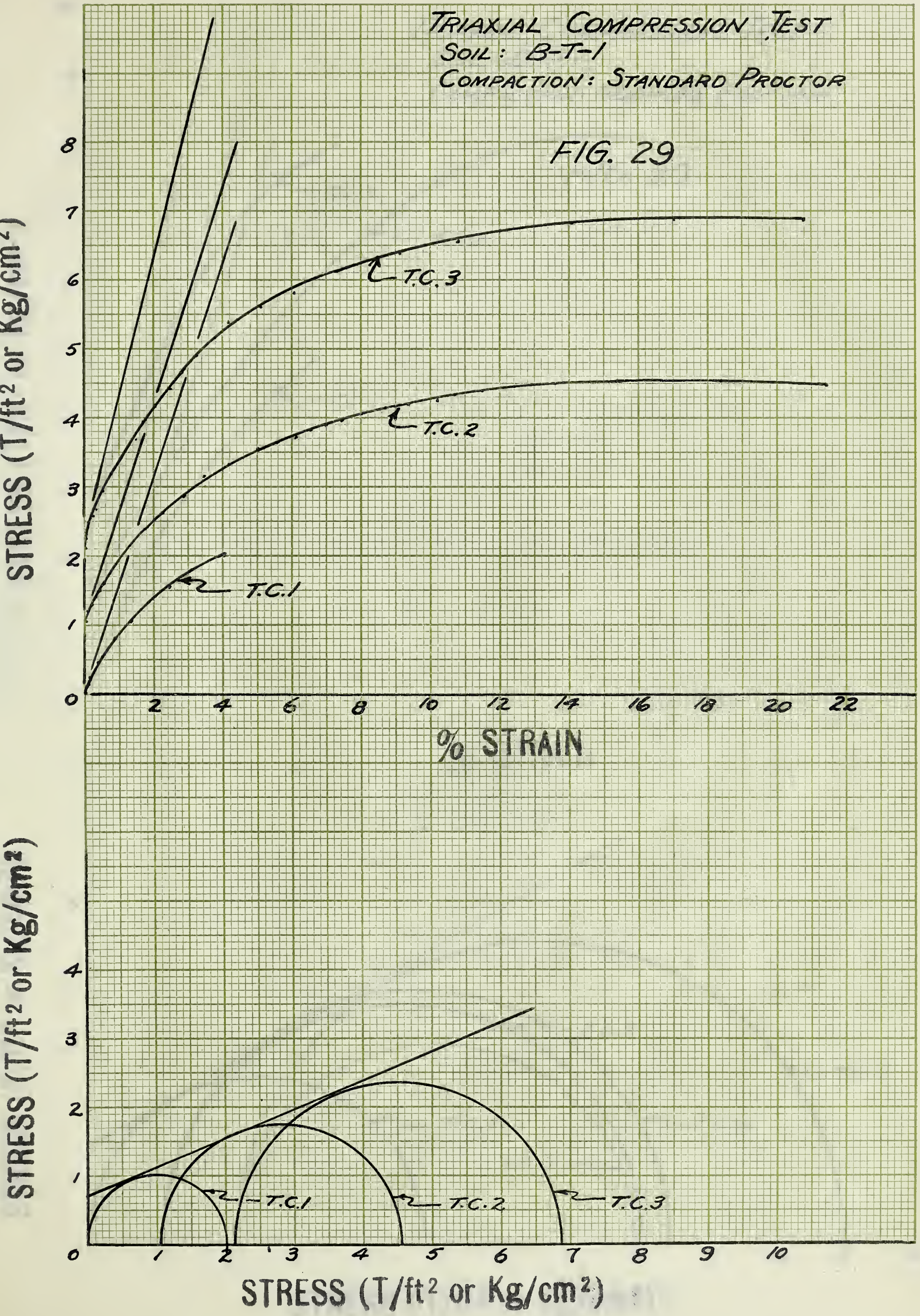

TRIAXIAL COMPRESSION TEST
SOIL : A-T-1
COMPACTION: MODIFIED PROCTOR
FIG. 26
T.C. 2
T.C. 1
STRESS (T/ft² or Kg/cm²)
% STRAIN
0
2
4
6
8
10
12
14
16
1
2
3
4
5
6
7
8
9
10

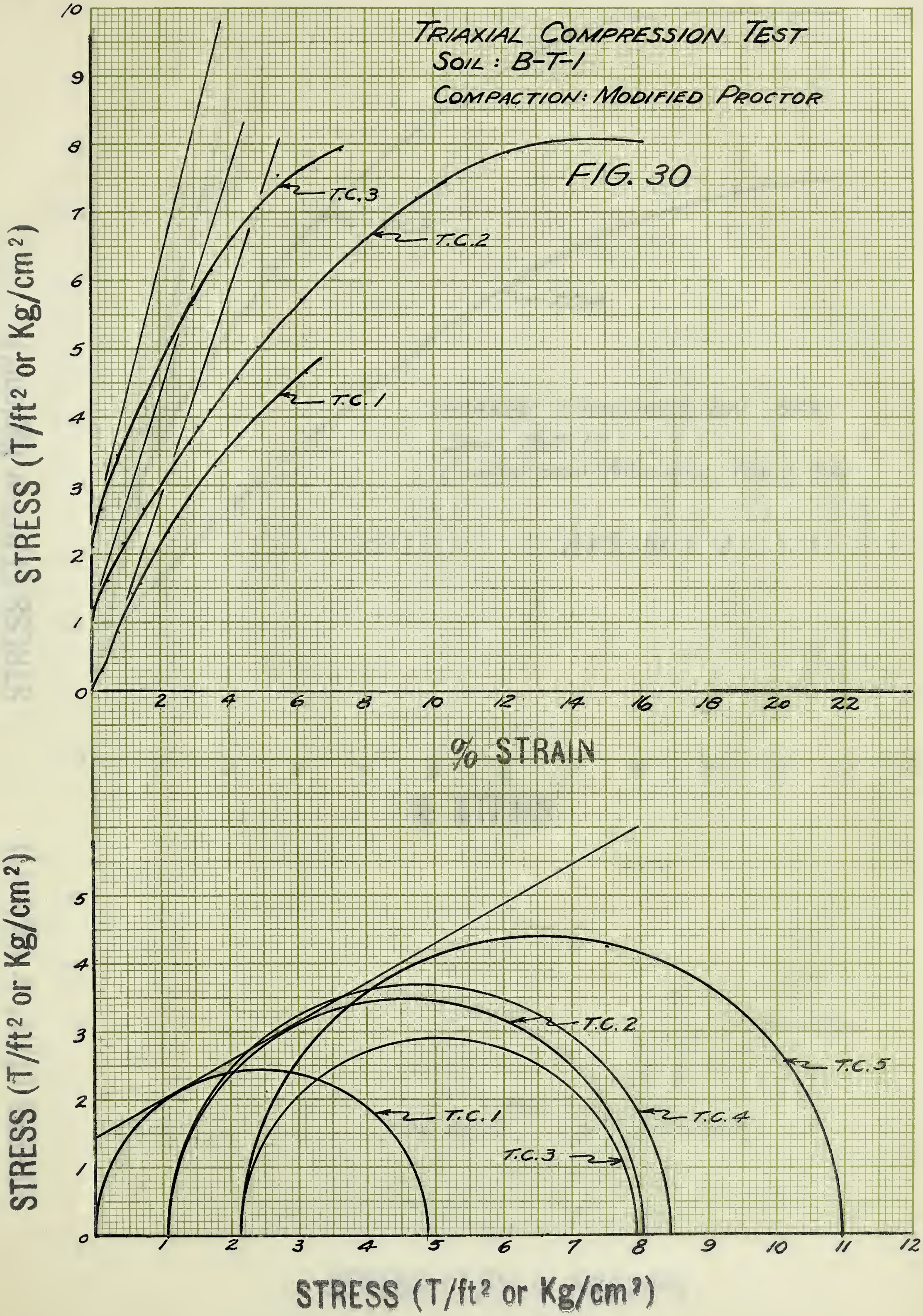

T.C. 1
T.C. 2
STRESS (T/ft² or Kg/cm²)
STRESS (T/ft² or Kg/cm²)
0
1
2
3
4
5
6
7
8
9
10

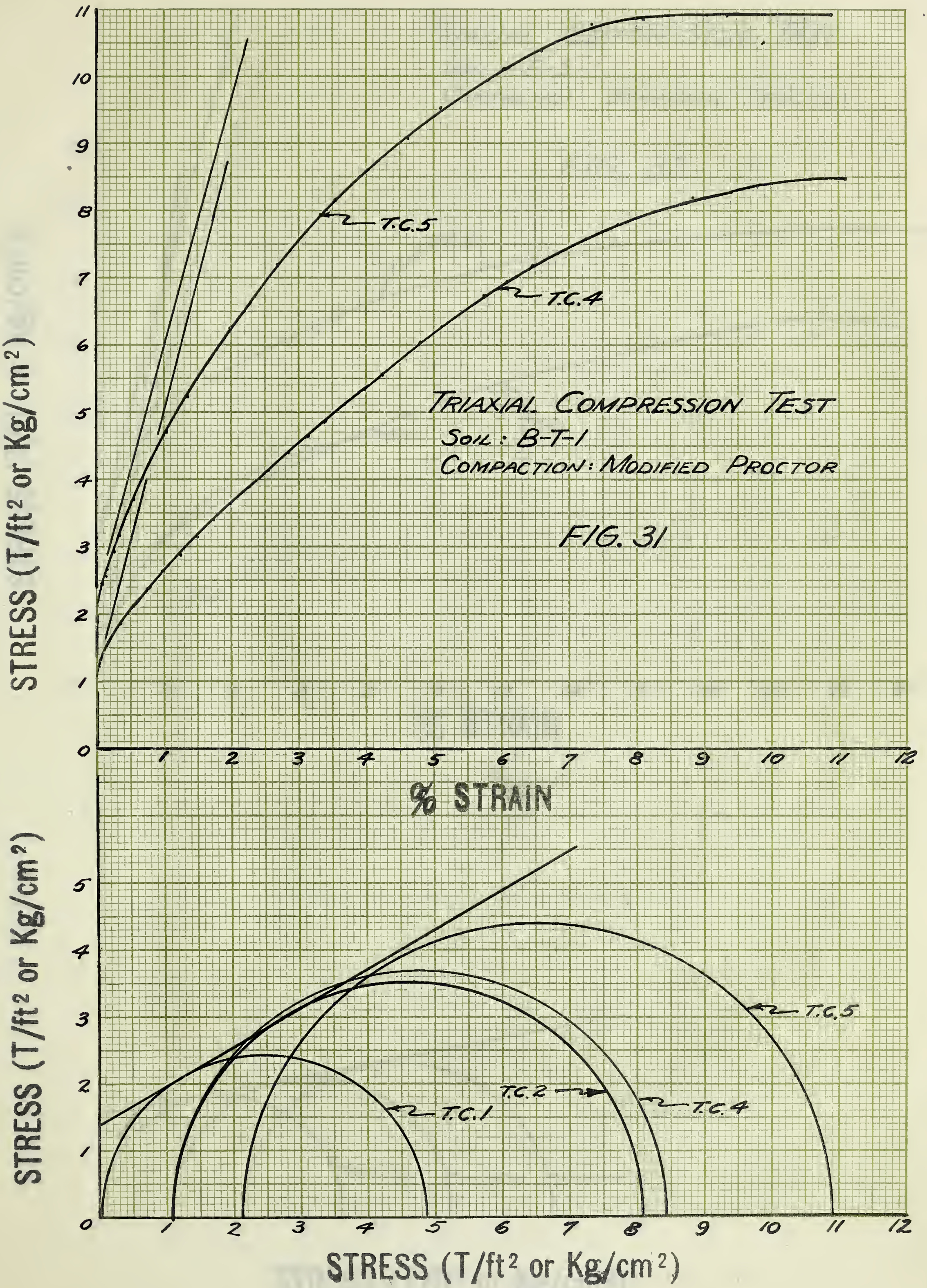

TRIAXIAL COMPRESSION TEST
SOIL : A-S-1
COMPACTION: STANDARD PROCTOR
FIG. 27
STRESS (T/ft² or Kg/cm²)
0
1
2
3
4
5
6
7
T.C.3
T.C.2
T.C.1
2
4
6
8
10
12
14
16
18
20
22
24
% STRAIN
STRESS (T/ft² or Kg/cm²)
0
1
2
3
T.C.2
T.C.3
T.C.1
1
2
3
4
5
6
STRESS (T/ft² or Kg/cm²)

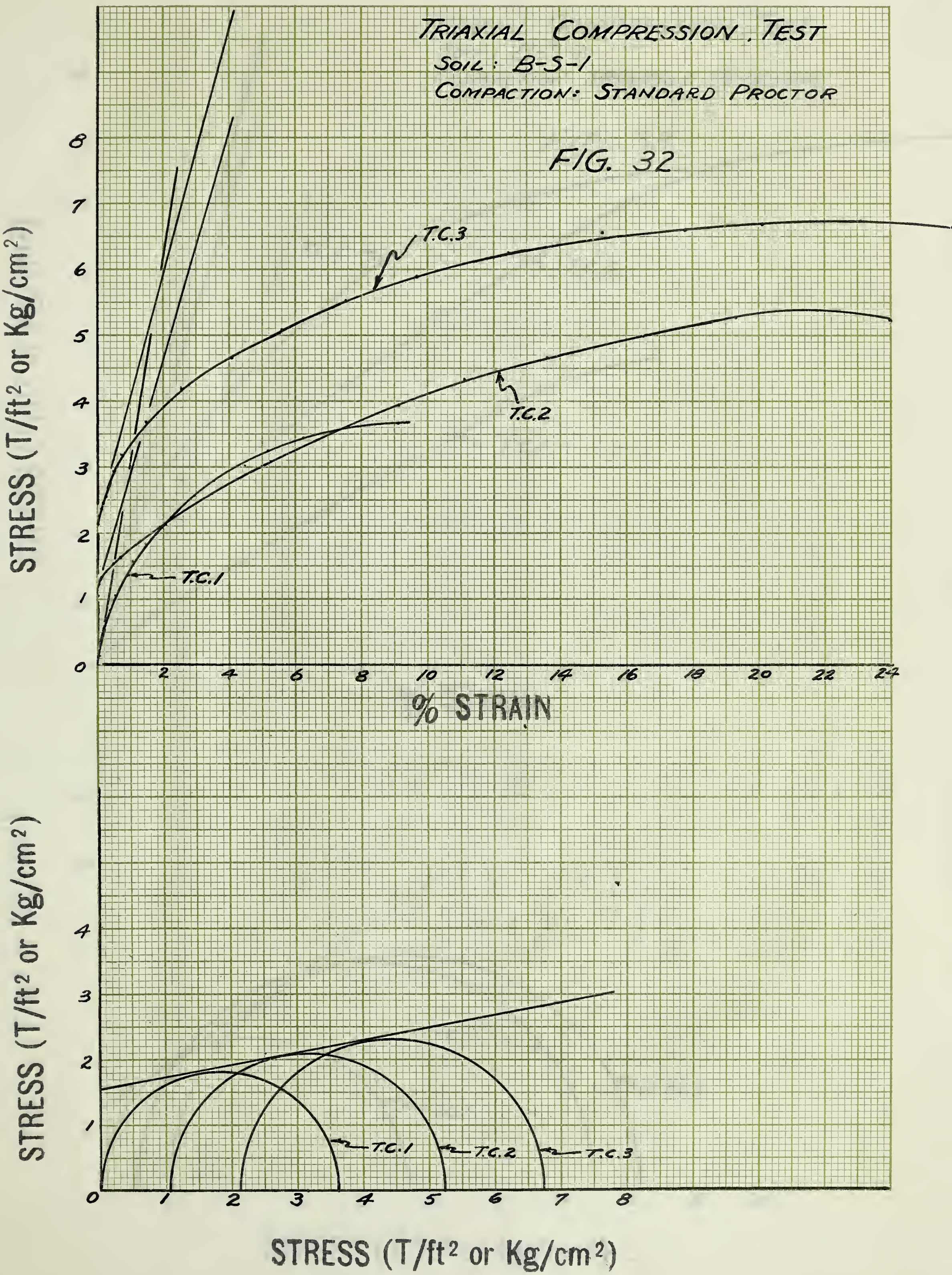

TRIAXIAL COMPRESSION TEST
SOIL : A-S-1
COMPACTION : MODIFIED PROCTOR
FIG. 28
T.C.3
T.C.2
T.C.1
STRESS (T/ft² or Kg/cm²)
% STRAIN
0
1
2
3
4
5
6
7
8
9
10
STRESS (T/ft² or Kg/cm²)
T.C.1
T.C.2
T.C.3
STRESS (T/ft² or Kg/cm²)

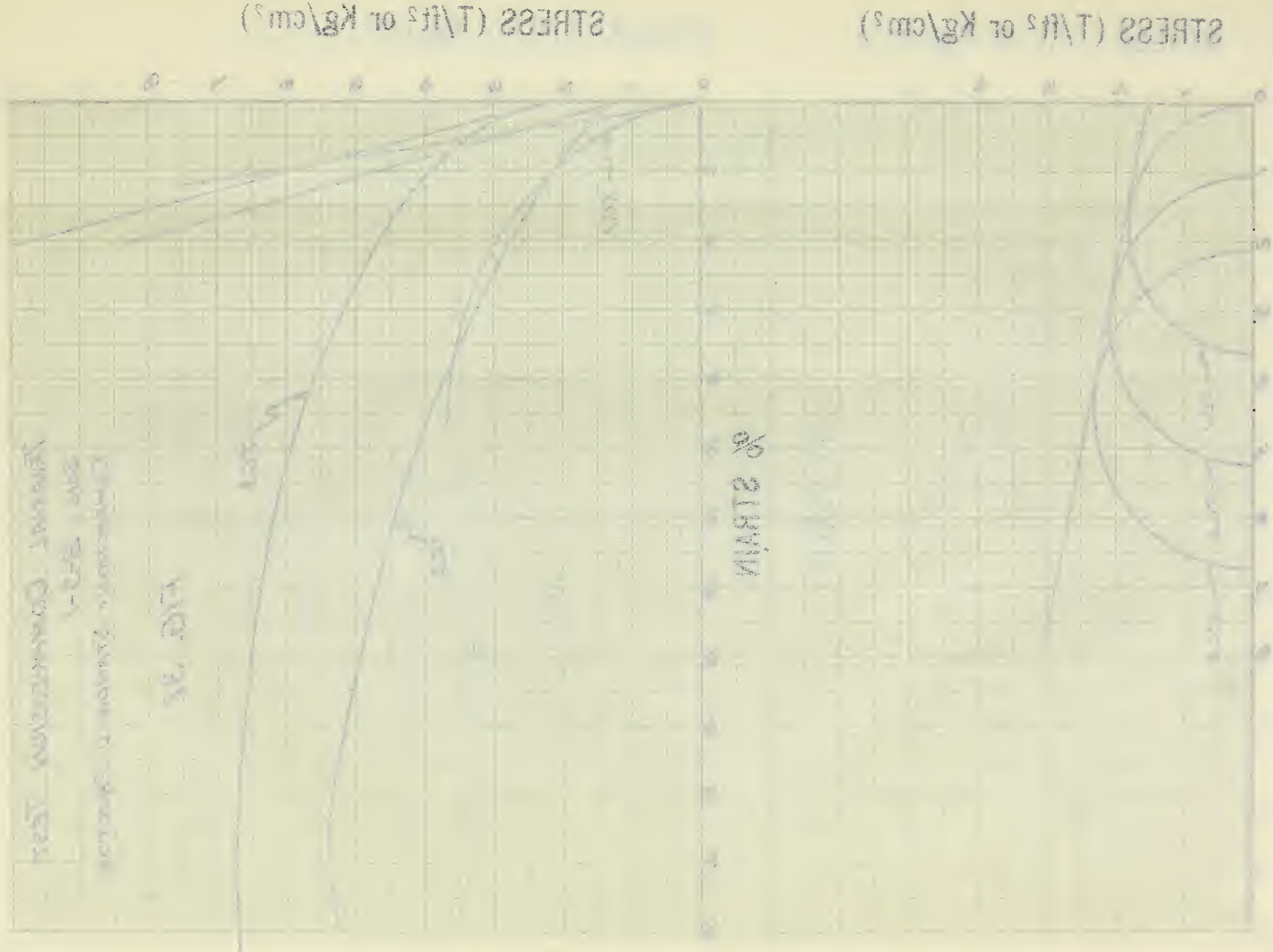


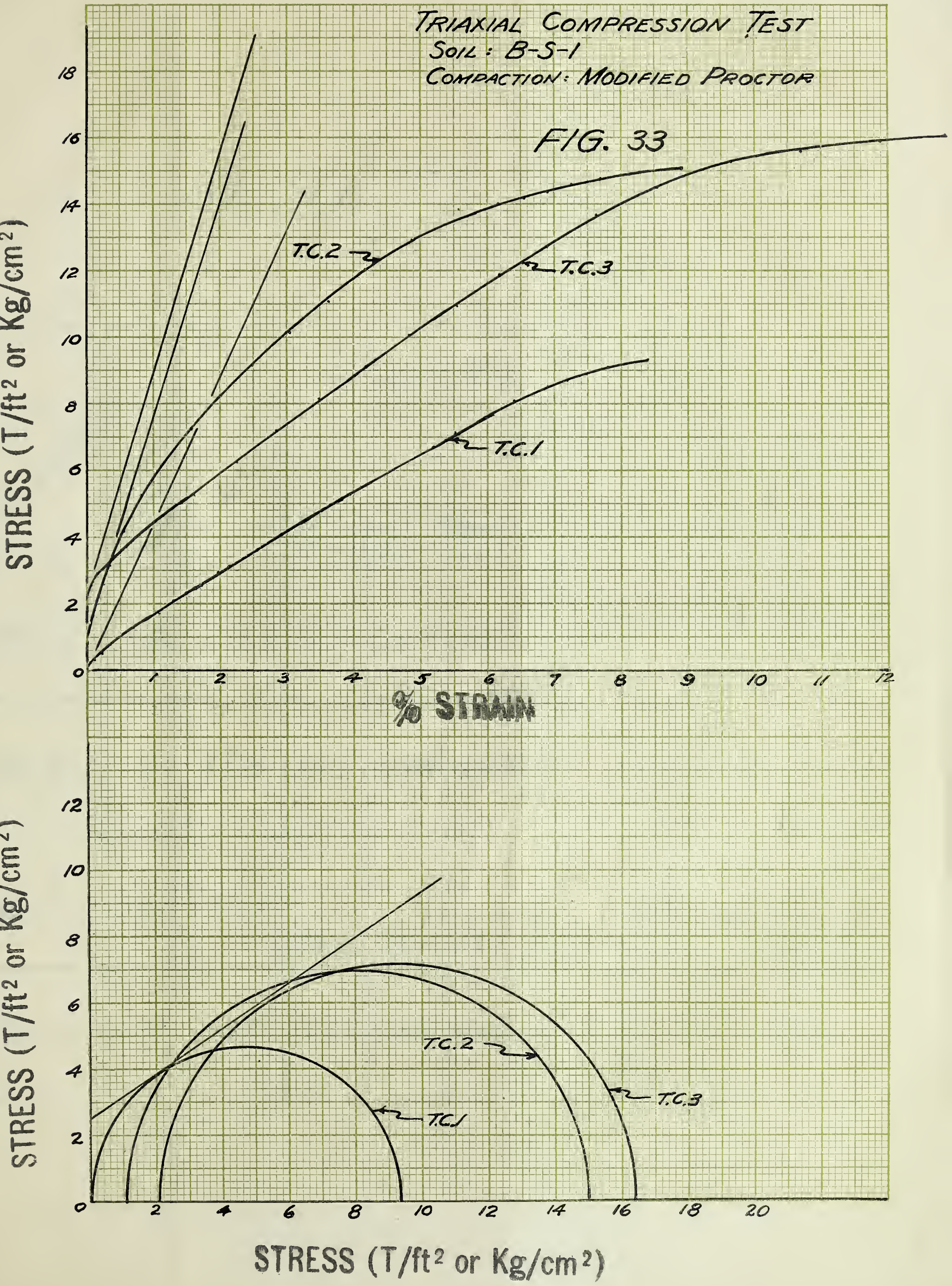

TRIAXIAL COMPRESSION TEST
SOIL: B-T-1
COMPACTION: STANDARD PROCTOR
FIG. 29
STRESS (T/ft² or Kg/cm²)
T.C.3
T.C.2
T.C.1
% STRAIN
STRESS (T/ft² or Kg/cm²)
T.C.1
T.C.2
T.C.3
STRESS (T/ft² or Kg/cm²)

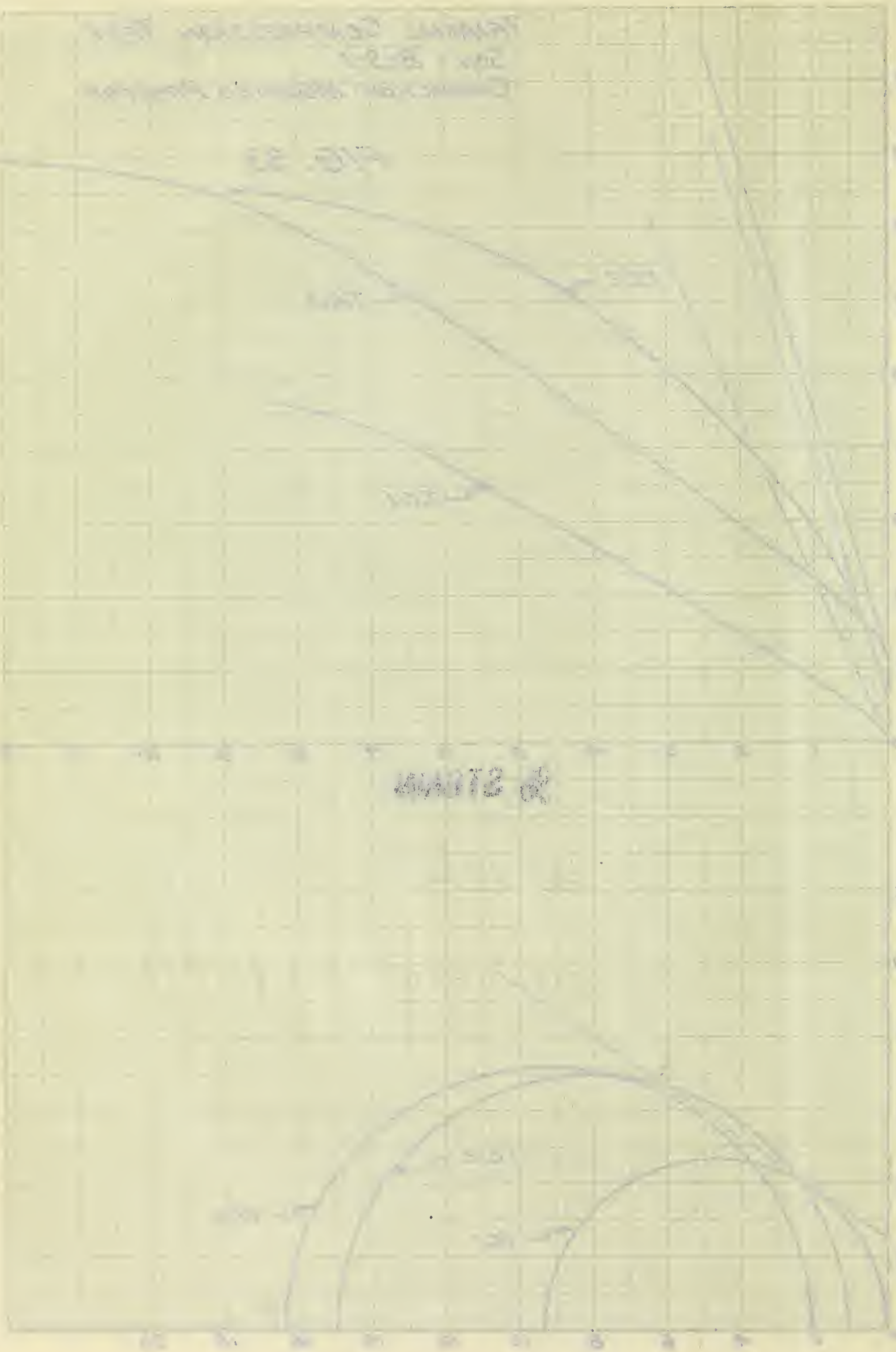

TRIAXIAL COMPRESSION TEST
SOIL : B-T-1
COMPACTION: MODIFIED PROCTOR
FIG. 30
T.C.3
T.C.2
T.C.1
STRESS (T/ft² or Kg/cm²)
% STRAIN
T.C.2
T.C.5
T.C.1
T.C.4
T.C.3
STRESS (T/ft² or Kg/cm²)
STRESS (T/ft² or Kg/cm²)


TRIAXIAL COMPRESSION TEST
SOIL: B-T-1
COMPACTION: MODIFIED PROCTOR
FIG. 31
T.C.5
T.C.4
STRESS (T/ft² or Kg/cm²)
% STRAIN
0
1
2
3
4
5
6
7
8
9
10
11
12
T.C.1
T.C.2
T.C.4
T.C.5
STRESS (T/ft² or Kg/cm²)
STRESS (T/ft² or Kg/cm²)


TRIAXIAL COMPRESSION TEST
SOIL: B-S-1
COMPACTION: STANDARD PROCTOR
FIG. 32
T.C.3
T.C.2
T.C.1
STRESS (T/ft² or Kg/cm²)
% STRAIN
0
2
4
6
8
10
12
14
16
18
20
22
24
1
2
3
4
5
6
7
8
STRESS (T/ft² or Kg/cm²)
T.C.1
T.C.2
T.C.3
STRESS (T/ft² or Kg/cm²)


STRESS (T/ft² or Kg/cm²)


TRIAXIAL COMPRESSION TEST
SOIL: B-S-1
COMPACTION: MODIFIED PROCTOR
FIG. 33
T.C.2
T.C.3
T.C.1
STRESS (T/ft² or Kg/cm²)
% STRAIN
0
1
2
3
4
5
6
7
8
9
10
11
12
14
16
18
20



TRIAXIAL COMPRESSION TEST RESULTS

Standard Proctor

Soil Number	Moisture Content W%	Density lbs. per cu. ft.	C	φ	Lateral Pressure 0 – W% (end)	Deg. of Sat.	Mod. of Def.	$\sigma_1 - \sigma_3$	Lateral Pressure 15 – W% (end)	Deg. of Sat.	Mod. of Def.	$\sigma_1 - \sigma_3$	Lateral Pressure 30 – W% (end)	Deg. of Sat.	Mod. of Def.	$\sigma_1 - \sigma_3$
A-T-1	14.3	105.4	0.58	22.3	14.1	0.69	374	1.60	14.1	0.65	422	4.[illegible]1	14.[illegible]	0.62	310	4.20
A-S-1	7.7	108.0	0.43	23.3	17.7	0.93	283	1.26	17.5	0.90	315	3.02	17.5	0.90	204	3.80
B-T-1	32.2	76.0	0.[illegible]	23.1	32.5	0.52	154	2.03	32.5	0.56	157	3.50	32.7	0.65	300	4.04
B-S-1	[illegible]	103.0	1.55	11.0	17.0	0.86	304	3.64	7.1	0.7[illegible]	17.4	4.19	17.2	0.86	188	4.70

Modified Proctor

Soil Number	Moisture Content W%	Density lbs. per cu. ft.	C	φ	Lateral Pressure 0 – W% (end)	Deg. of Sat.	Mod. of Def.	$\sigma_1 - \sigma_3$	Lateral Pressure 15 – W% (end)	Deg. of Sat.	Mod. of Def.	$\sigma_1 - \sigma_3$	Lateral Pressure 30 – W% (end)	Deg. of Sat.	Mod. of Def.	$\sigma_1 - \sigma_3$
A-T-1	12.1	113.4	0.95	29.3	11.7	0.63	360	3.11					11.7	0.54	3[illegible]4	7.07
A-S-1	13.2	114.1	1.39	35.8	13.0	0.74	447	5.37	12.9	0.80	482	8.20	13.1	0.78	520	7.70
B-T-1	26.0 26.8	85.4 83.4	1.43	30.1	26.6	0.91	151	4.87	25.2 27.0	0.83 0.[illegible]4	39[illegible] 161	7.40 5.[illegible]0	25.2 26.5	0.5[illegible] 0.85	372 202	8.00 5.50
B-S-1	12.2	114.6	2.5	34.6	12.0	0.67	443	9.35	11.9	0.73	645	13.9[illegible]	12.2	[illegible]	670	14.3[illegible]

FIG. 34

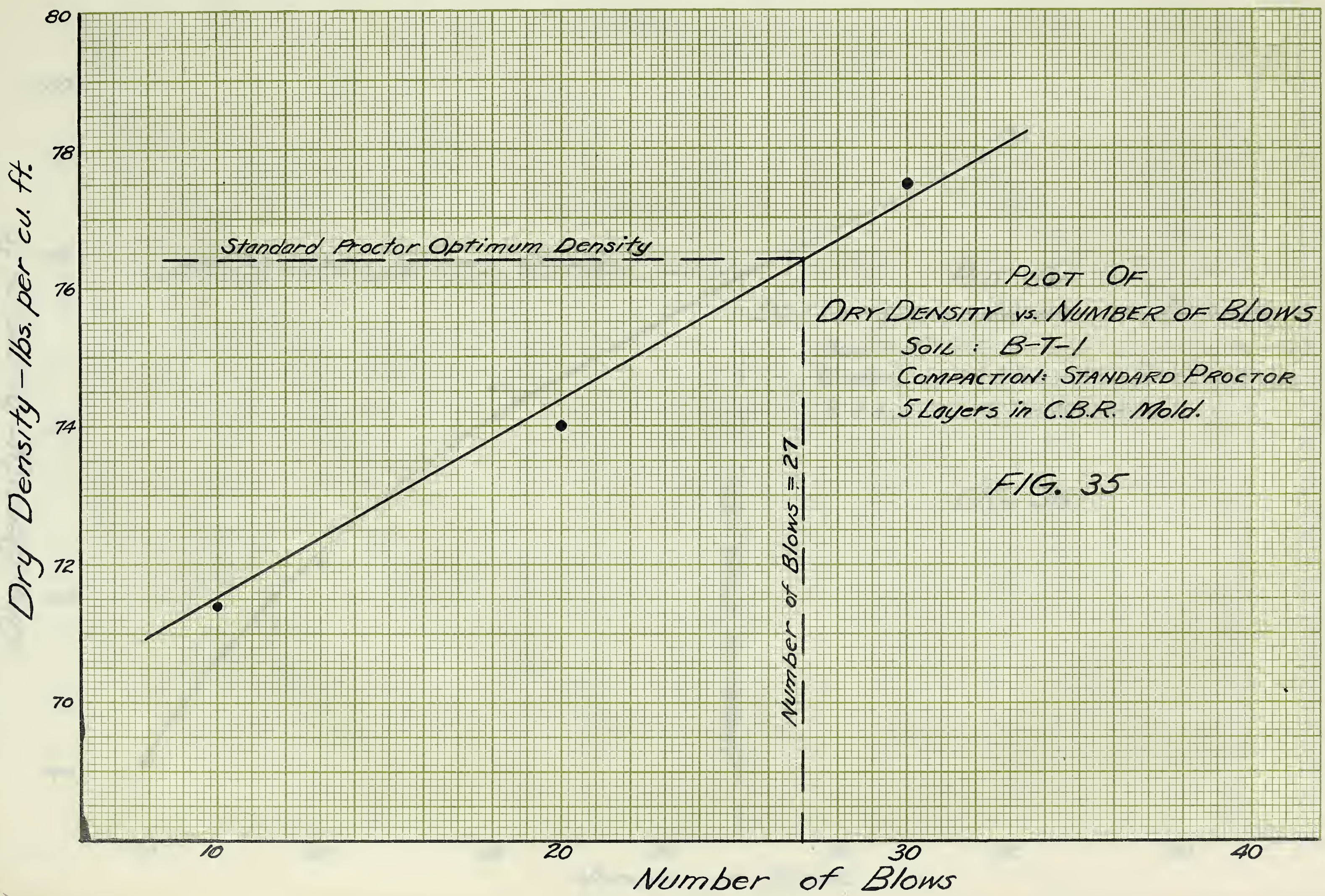

Plot of
Dry Density vs. Number of Blows
Soil : B-T-1
Compaction: Standard Proctor
5 Layers in C.B.R. Mold.
Standard Proctor Optimum Density
Number of Blows = 27
Dry Density – lbs. per cu. ft.
80
78
76
74
72
70
Number of Blows
10
20
30
40

FIG. 35

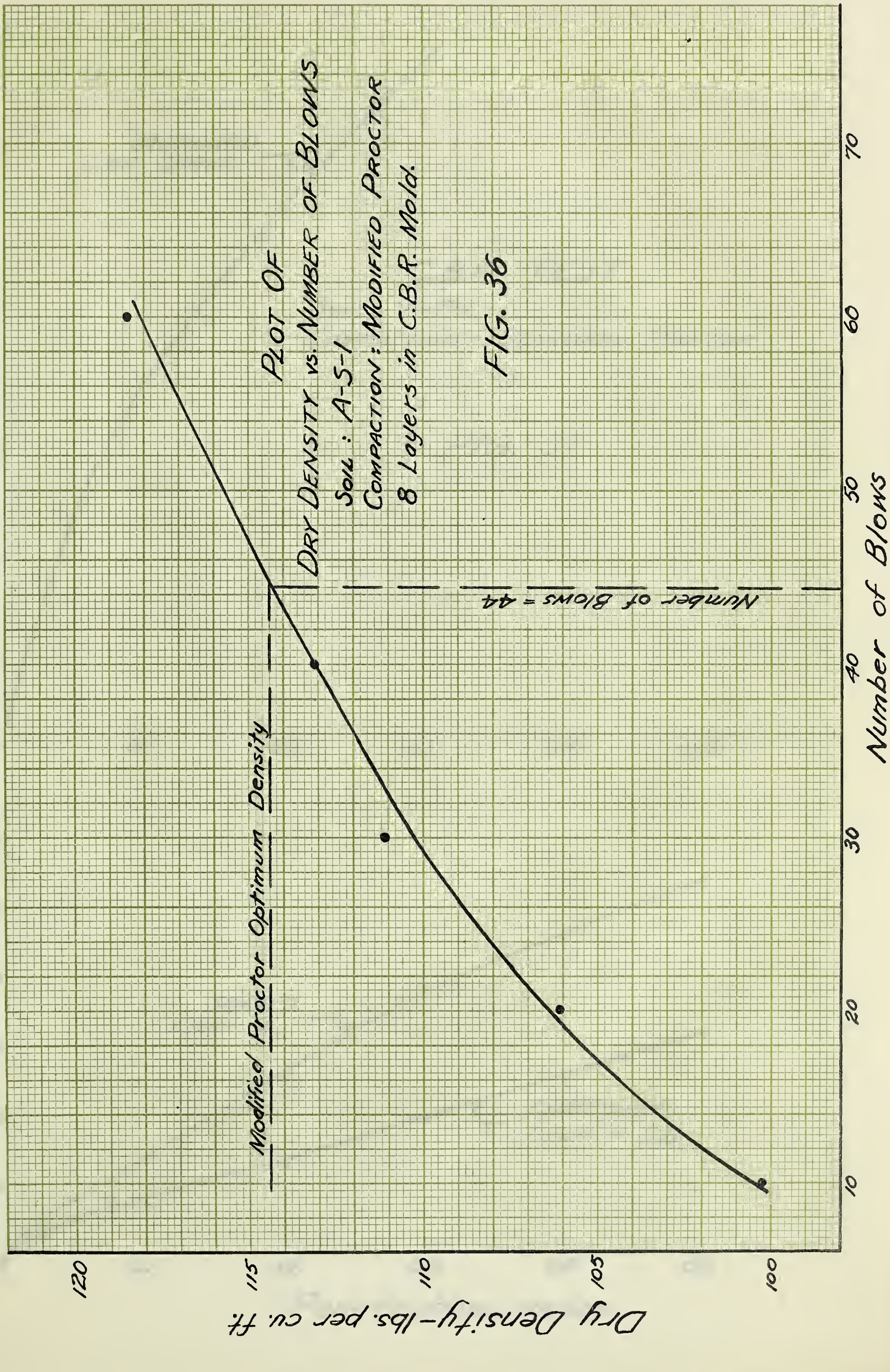
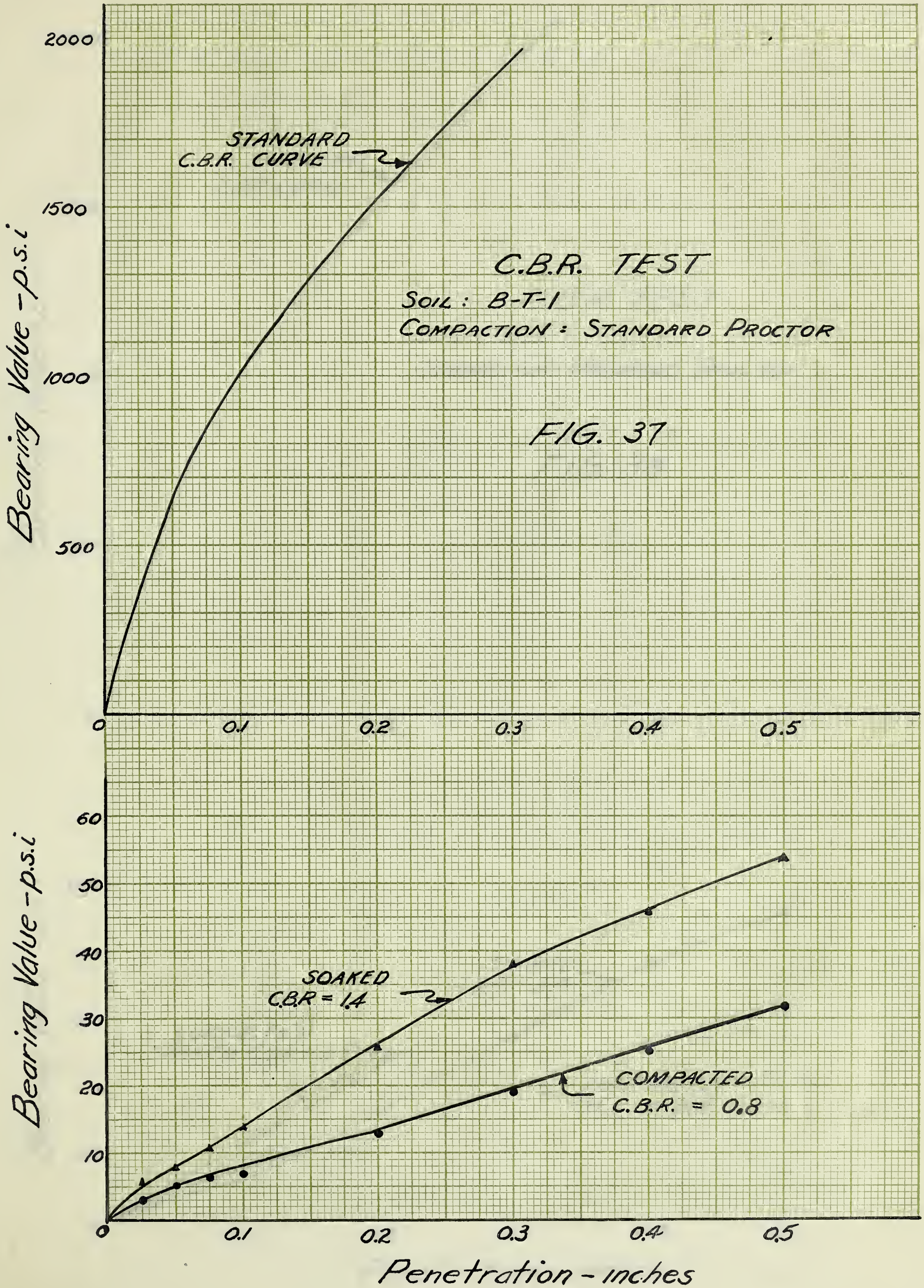
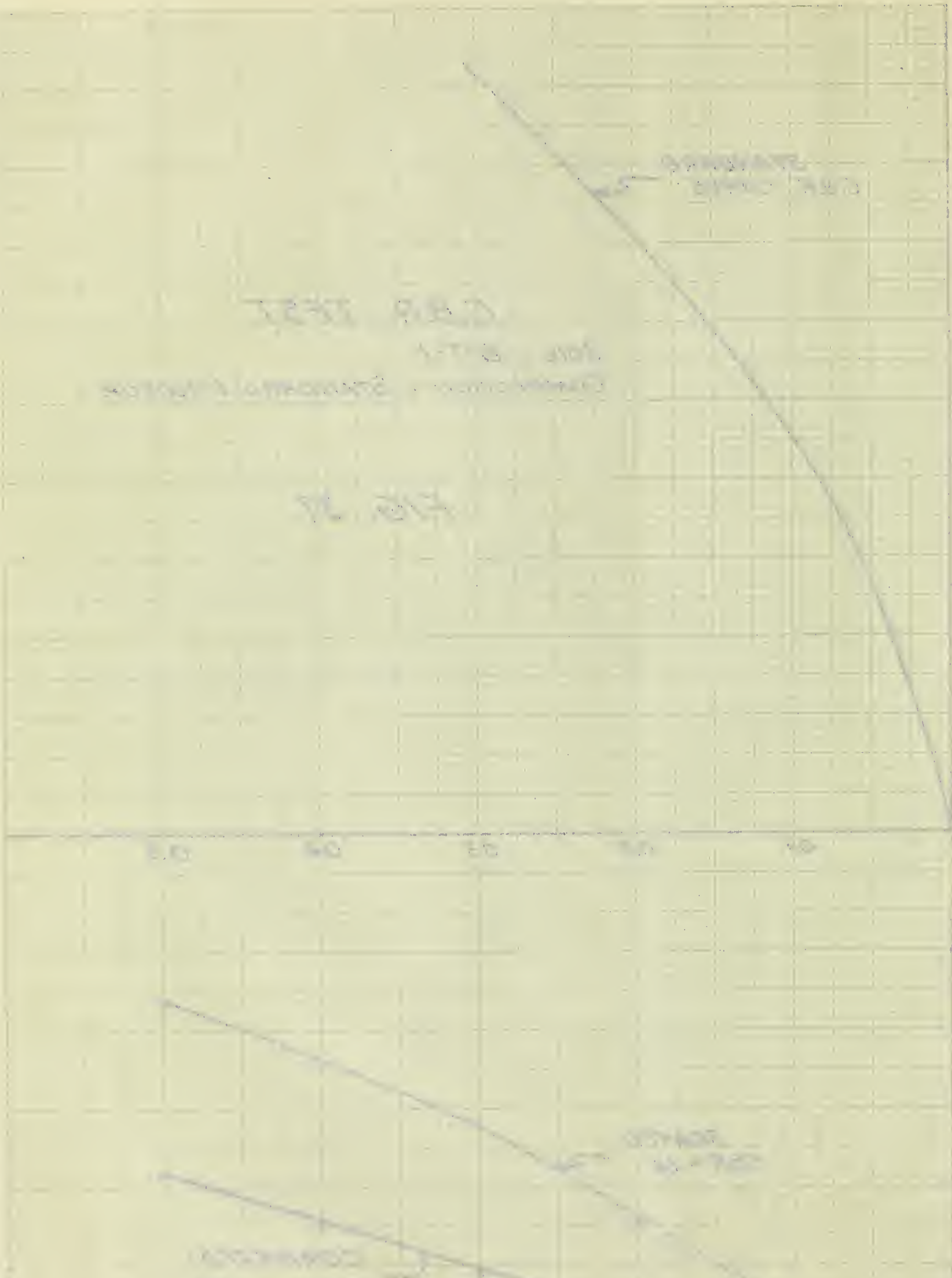

Plot of
Dry Density vs. Number of Blows
Soil : A-5-1
Compaction : Modified Proctor
8 Layers in C.B.R. Mold.
Modified Proctor Optimum Density
Number of Blows = 44
Number of Blows
10
20
30
40
50
60
70
Dry Density - lbs. per cu. ft.
100
105
110
115
120

FIG. 36



2000
1500
1000
500
0
0.1
0.2
0.3
0.4
0.5
Bearing Value - p.s.i
STANDARD C.B.R. CURVE
C.B.R. TEST
SOIL: B-T-1
COMPACTION: STANDARD PROCTOR
FIG. 37
60
50
40
30
20
10
Bearing Value - p.s.i
SOAKED C.B.R = 1.4
COMPACTED C.B.R. = 0.8
Penetration - inches



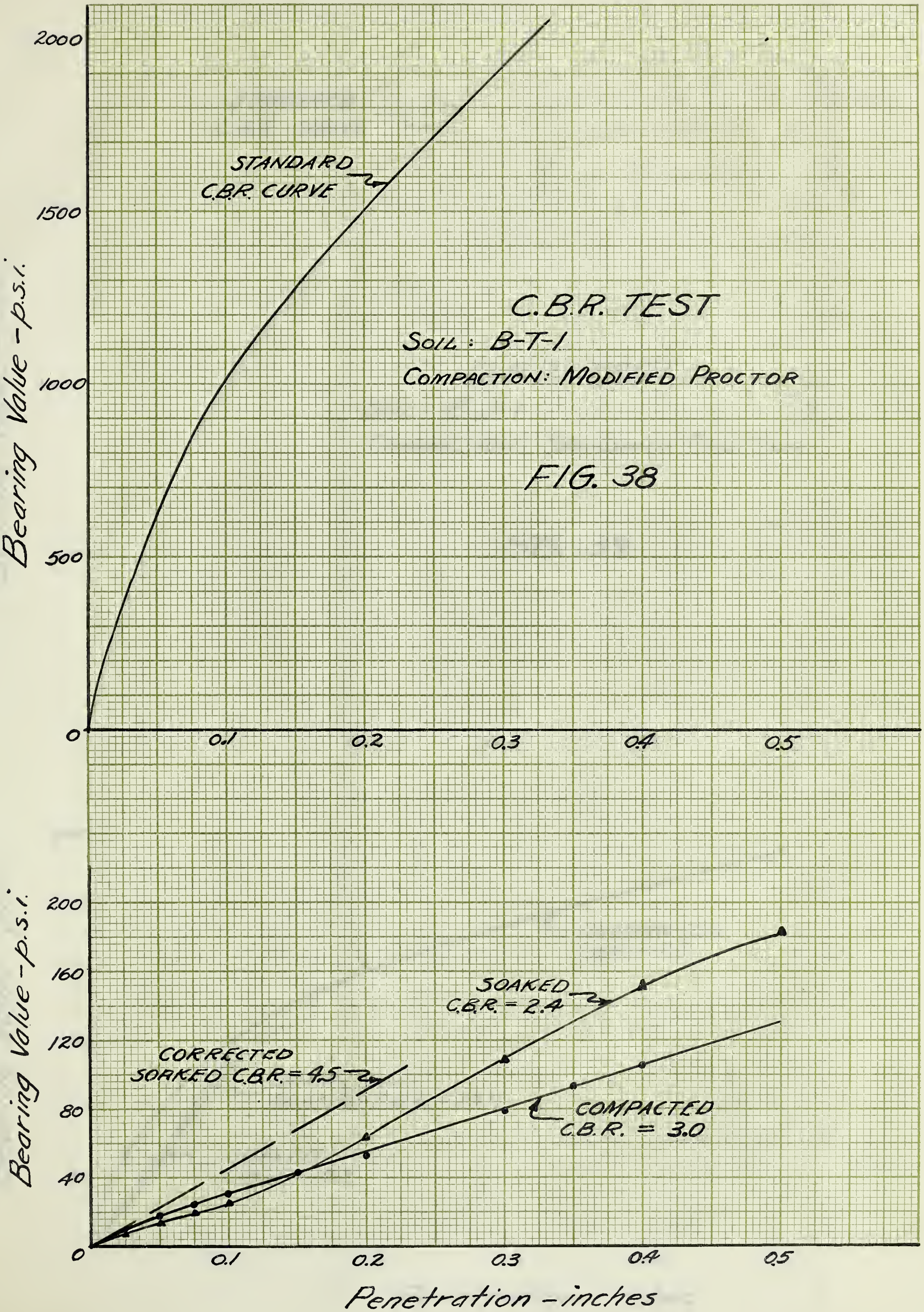
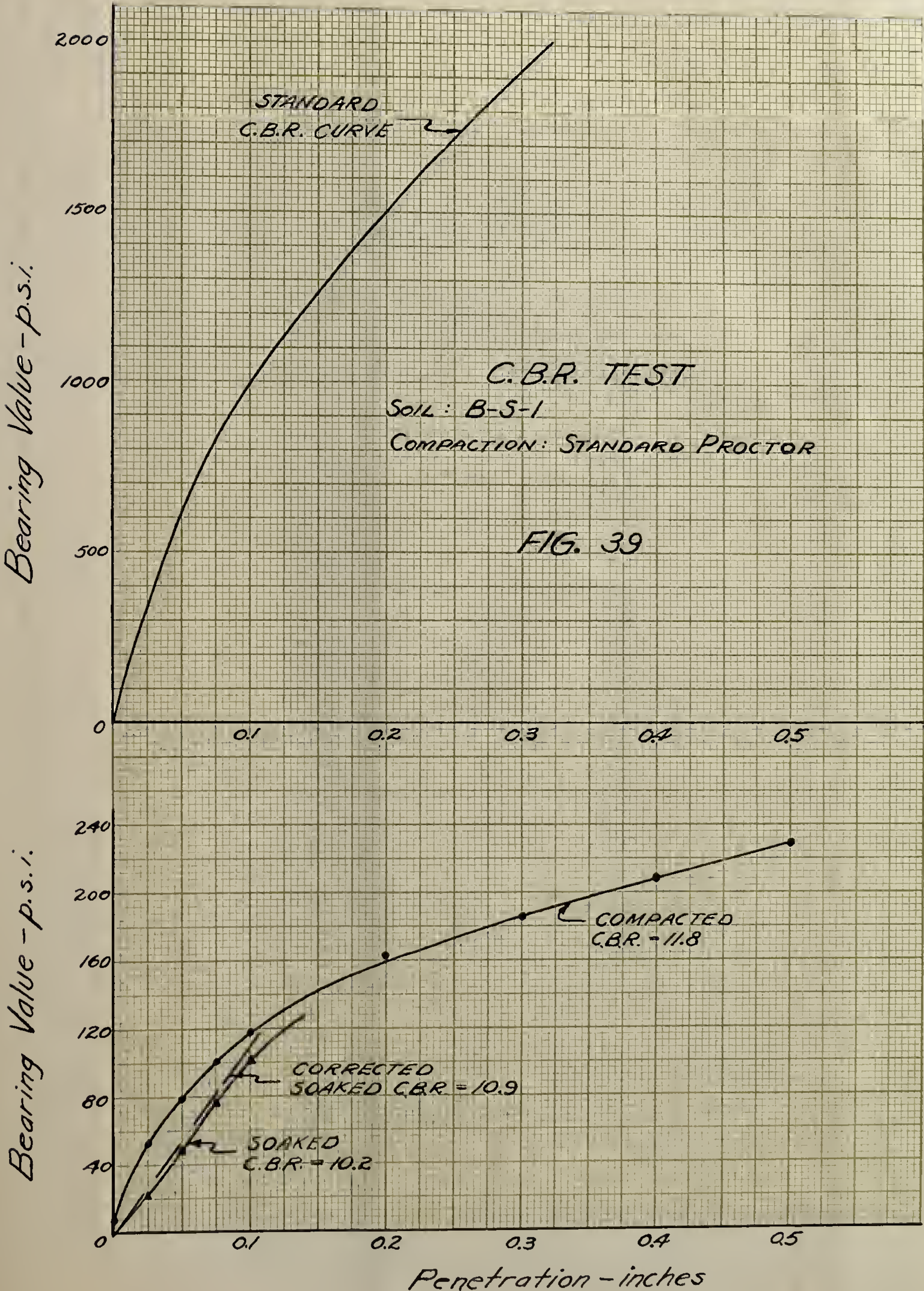

C.B.R. TEST
SOIL : B-T-1
COMPACTION: MODIFIED PROCTOR
STANDARD C.B.R. CURVE
SOAKED C.B.R. = 2.4
CORRECTED SOAKED C.B.R. = 4.5
COMPACTED C.B.R. = 3.0
Bearing Value - p.s.i.
Penetration - inches
2000
1500
1000
500
0
200
160
120
80
40
0.1
0.2
0.3
0.4
0.5

FIG. 38


2000
1500
1000
500
0
STANDARD
C.B.R. CURVE
C.B.R. TEST
SOIL : B-S-1
COMPACTION : STANDARD PROCTOR
FIG. 39
Bearing Value - p.s.i.
0.1
0.2
0.3
0.4
0.5
240
200
160
120
80
40
0
COMPACTED
C.B.R. = 11.8
CORRECTED
SOAKED C.B.R. = 10.9
SOAKED
C.B.R. = 10.2
Bearing Value - p.s.i.
Penetration - inches

CALIFORNIA BEARING RATIO TEST RESULTS

	B-7-1		B-S-1
	STANDARD	MODIFIED	STANDARD
Moisture content for compaction	32.0	25.8	17.2
Moisture content after soaking	32.2	26.6	[illegible]
Moisture content (Top 1" layer)	32.5	29.1	[illegible]
Degree of saturation (Compacted sample)	[illegible]	[illegible]	[illegible]
Degree of saturation (Soaked sample)	[illegible]	92.9	90.0
Dry density (Compacted sample)	[illegible]	[illegible]	105.4
Dry density (Soaked sample)	50.4	[illegible]	[illegible]
Percentage expansion upon soaking	-0.2	[illegible]	0.9
C.B.R. of compacted sample	[illegible]	3.0	[illegible]
C.B.R. of soaked sample (Corrected)	1.4	4.5	[illegible]
C.B.R. of soaked sample (Uncorrected)	1.4	2.4	[illegible]

FIG. 40

FREEZING TEST RESULTS

	B-T-1				B-S-1	
	STANDARD PROCTOR		MODIFIED PROCTOR		STANDARD PROCTOR	
	COMPACTED SAMPLE	SOAKED SAMPLE FROM C.B.R. TEST	COMPACTED SAMPLE	SOAKED SAMPLE FROM C.B.R. TEST	COMPACTED SAMPLE	SOAKED SAMPLE FROM C.B.R. TEST
Moisture content - percent	34.1	32.2	26.4	26.6	18.5	20.1
Dry density - lbs. per cu. ft.	77.6	80.4	86.2	88.8	102.7	105.0
Degree of saturation - percent	87.0	90.0	84.0	92.9	75.5	90.0
Void ratio	0.937	0.860	0.755	0.686	0.643	0.601
Ultimate expansion - percent	-0.4	+0.9	0.0	+2.3	0.0	+0.5

FIG. 41

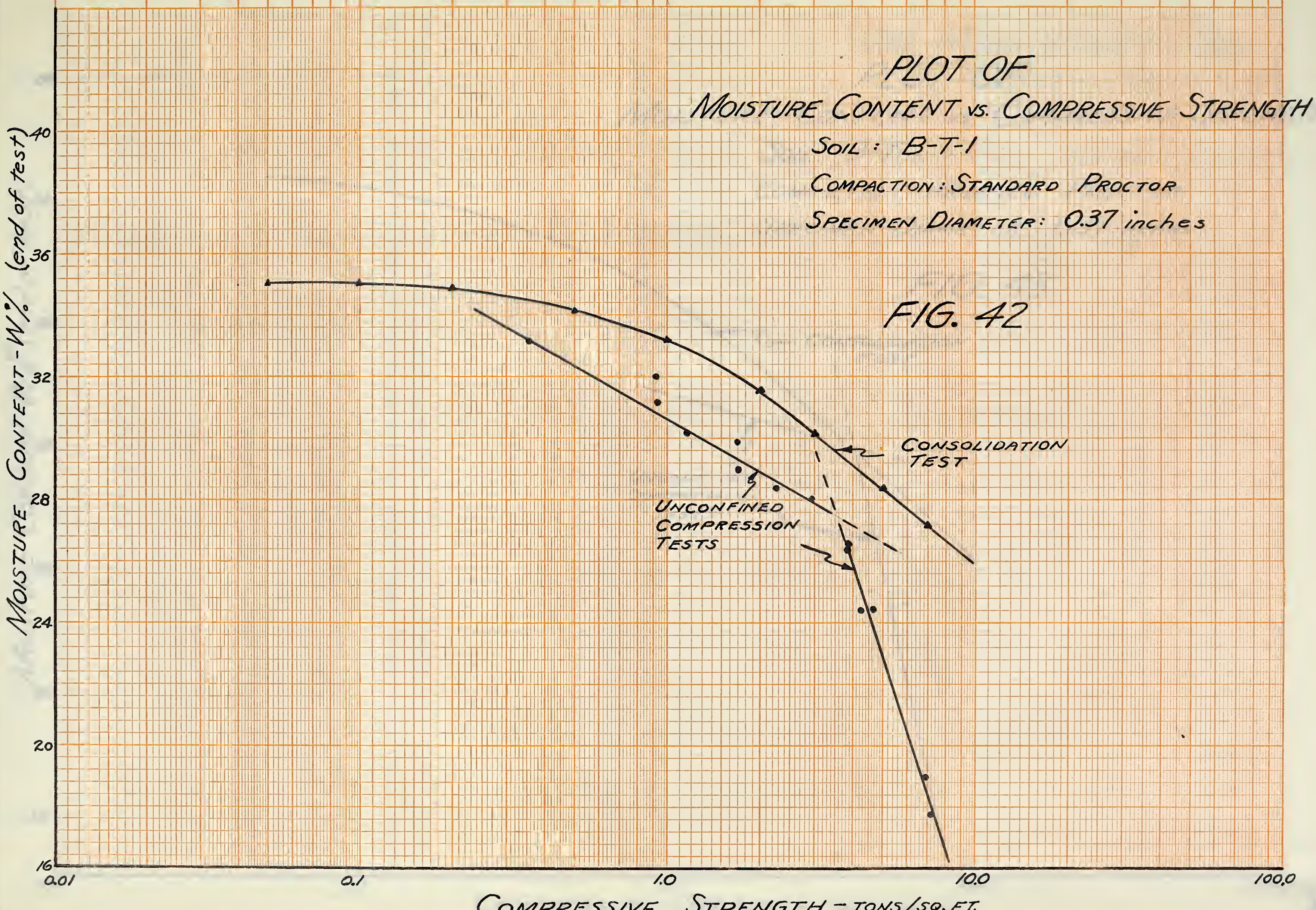
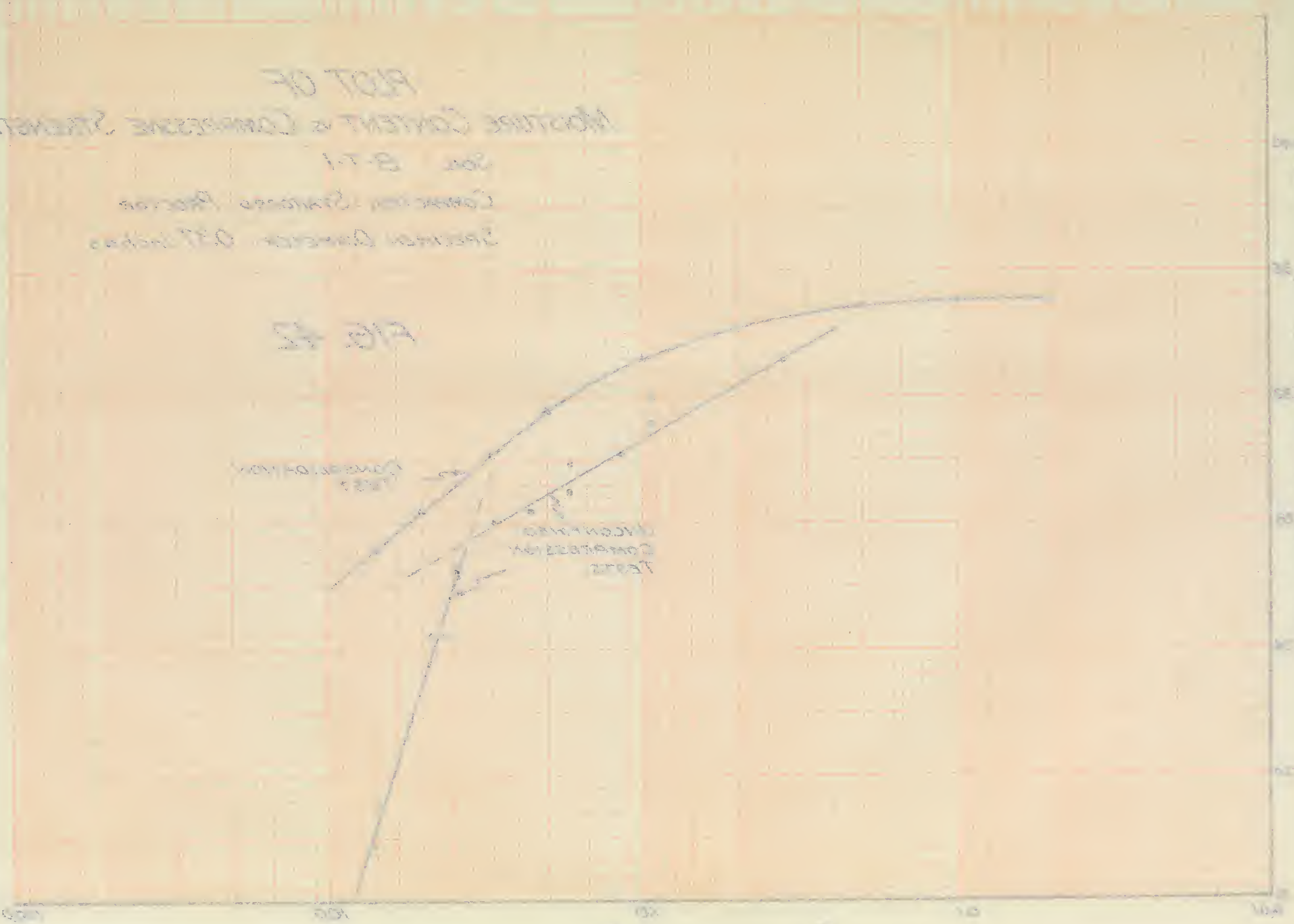
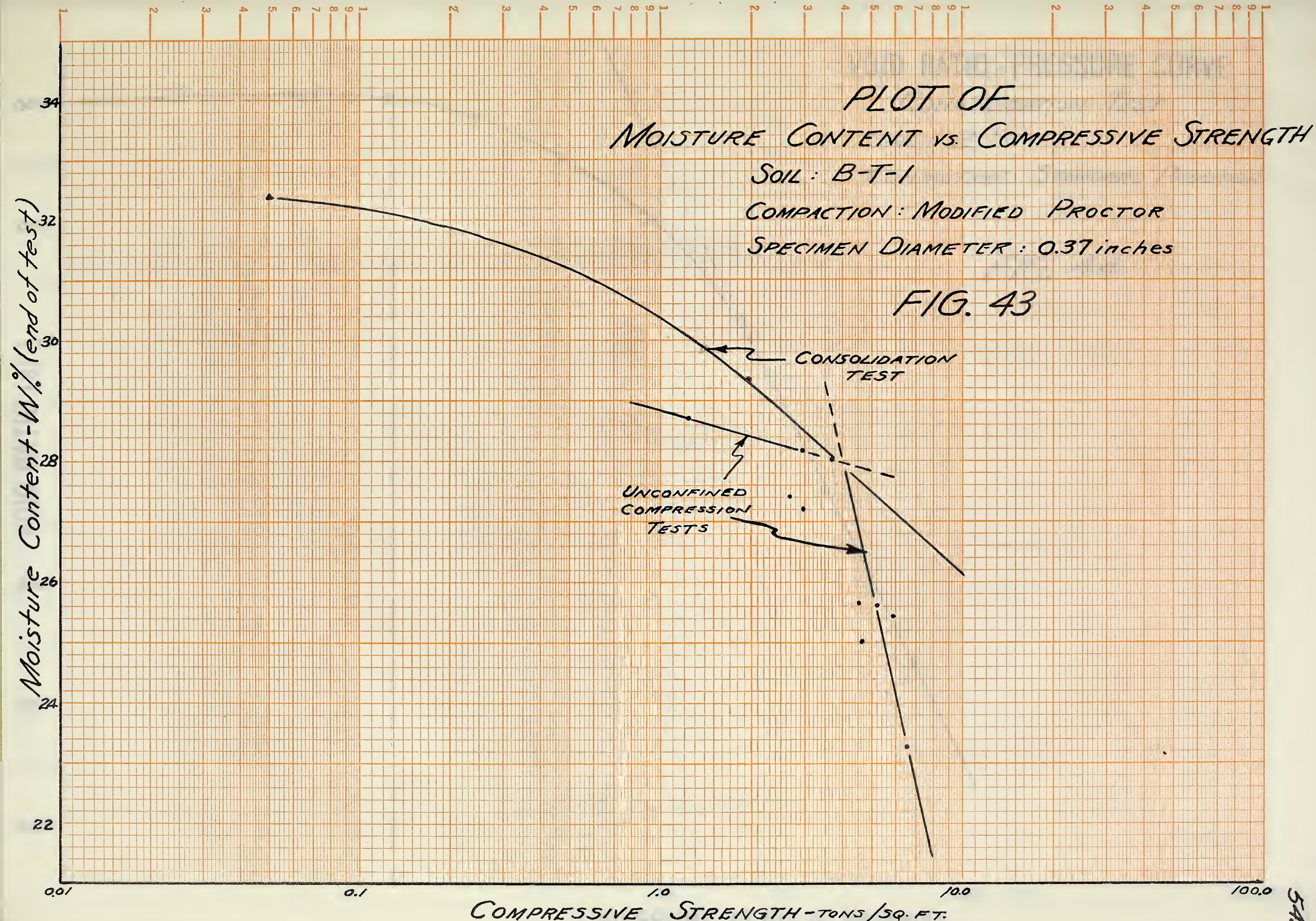
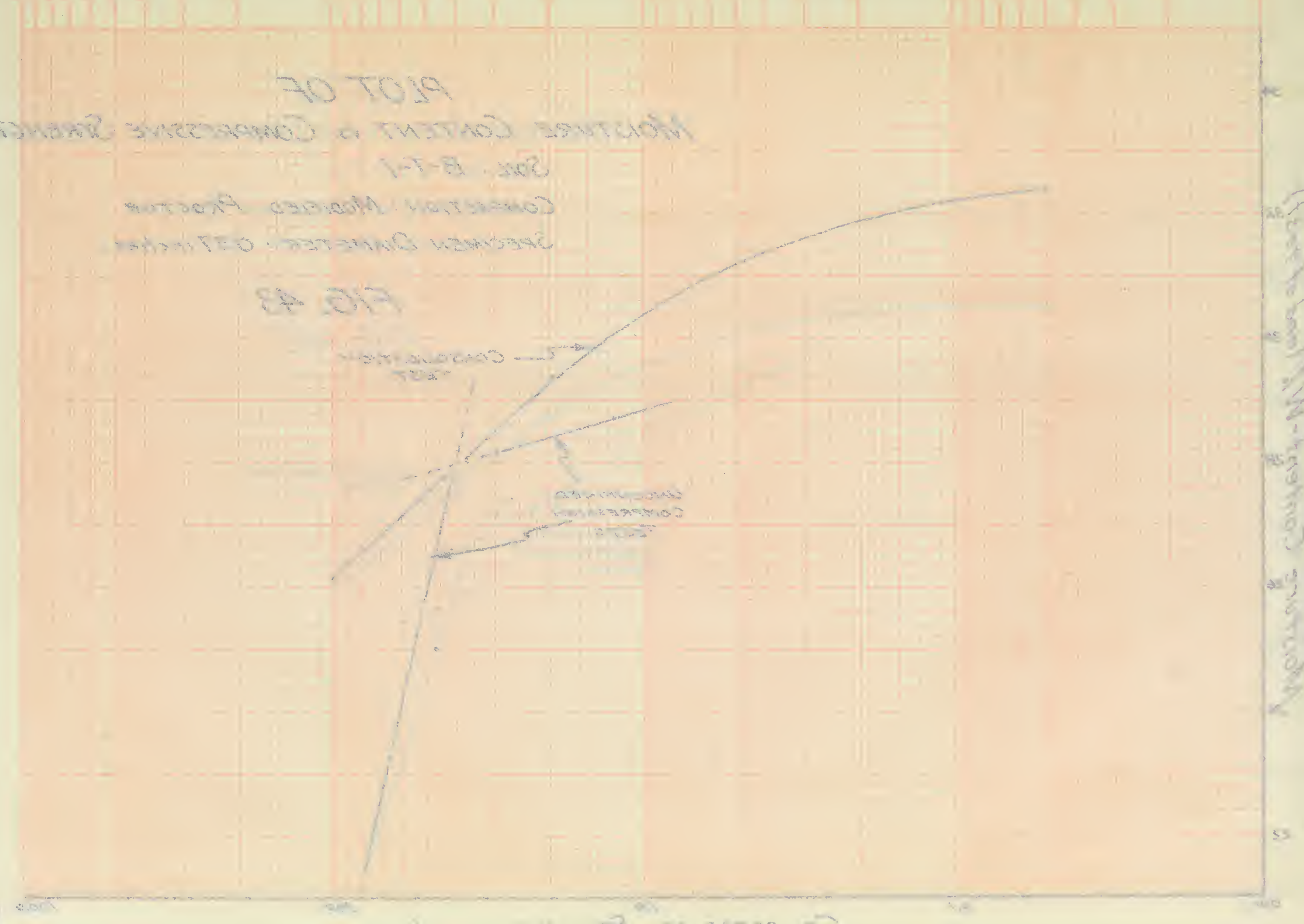
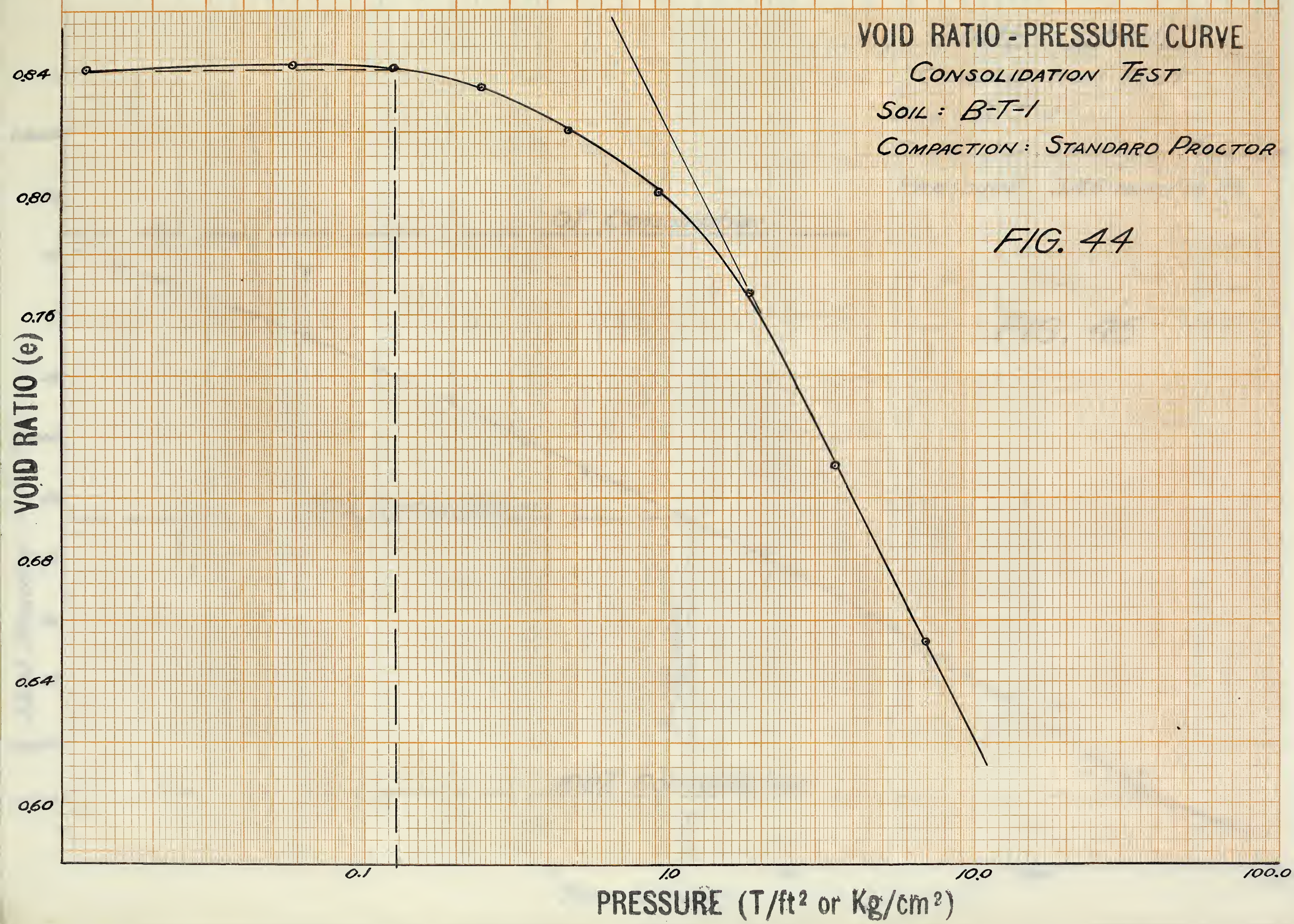

PLOT OF
MOISTURE CONTENT vs. COMPRESSIVE STRENGTH
SOIL : B-T-1
COMPACTION : STANDARD PROCTOR
SPECIMEN DIAMETER: 0.37 inches
CONSOLIDATION TEST
UNCONFINED COMPRESSION TESTS
MOISTURE CONTENT - W% (end of test)
16
20
24
28
32
36
40
0.01
0.1
1.0
10.0
100.0
COMPRESSIVE STRENGTH - TONS/SQ. FT.

FIG. 42


PLOT OF
Moisture Content vs Compressive Strength
Soil B-1-1
Compaction Standard Proctor
Specimen Diameter 0.17 Inches
FIG 42
Consolidation Tests
Unconfined Compression Tests

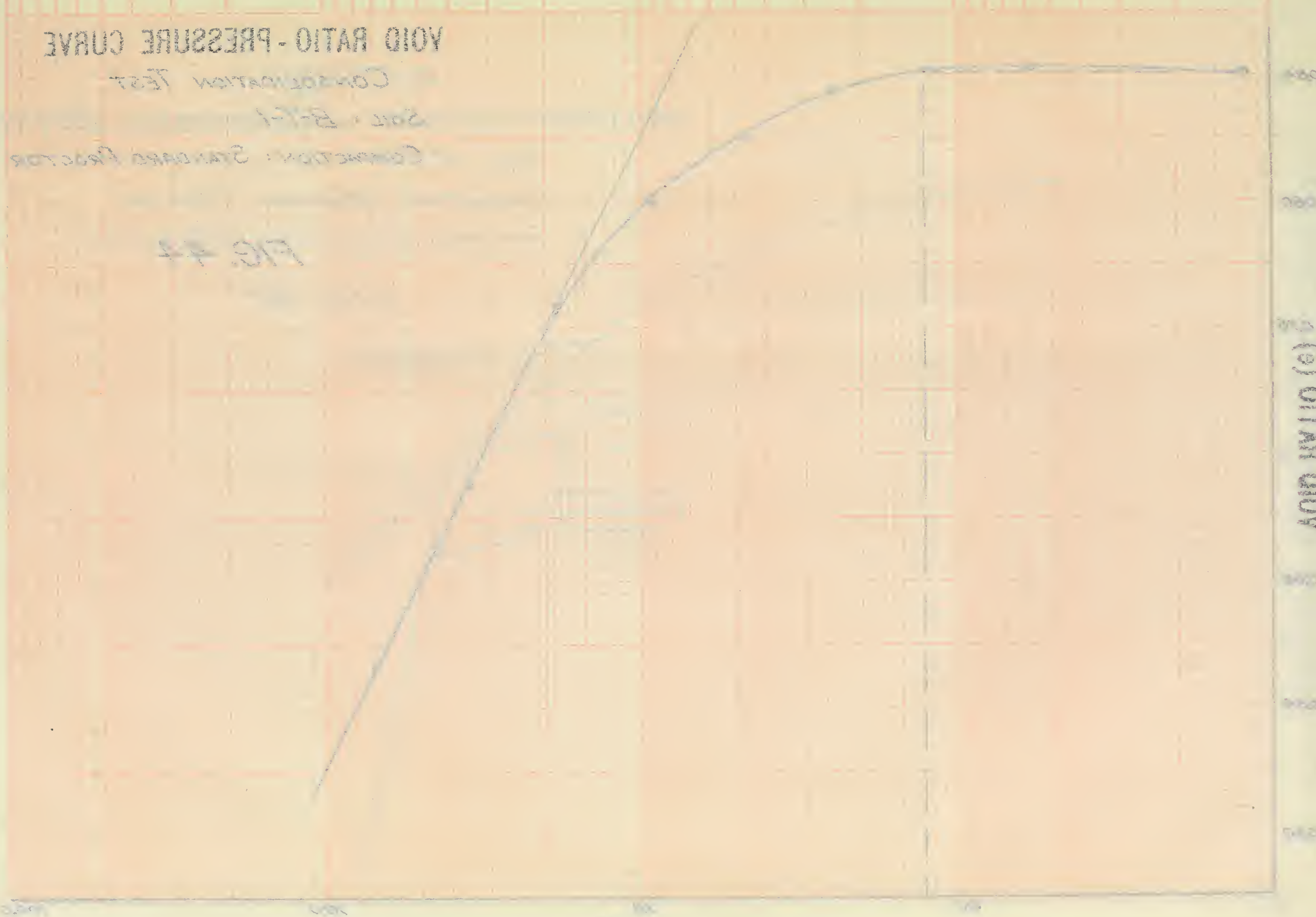

PLOT OF
MOISTURE CONTENT vs. COMPRESSIVE STRENGTH
SOIL: B-T-1
COMPACTION: MODIFIED PROCTOR
SPECIMEN DIAMETER: 0.37 inches
FIG. 43
CONSOLIDATION TEST
UNCONFINED COMPRESSION TESTS
Moisture Content-W% (end of test)
34
32
30
28
26
24
22
COMPRESSIVE STRENGTH-TONS/SQ. FT.
0.01
0.1
1.0
10.0
100.0


PLOT OF
Moisture Content vs Compressive Strength
Soil B-1-1
Compaction: Modified Proctor
FIG 43
Consolidation Test
Unconfined Compression Tests



VOID RATIO - PRESSURE CURVE
CONSOLIDATION TEST
SOIL : B-T-1
COMPACTION : STANDARD PROCTOR
0.84
0.80
0.76
0.68
0.64
0.60
VOID RATIO (e)
0.1
1.0
10.0
100.0
PRESSURE (T/ft² or Kg/cm²)

FIG. 44


VOID RATIO - PRESSURE CURVE
Consolidation Test
Soil: B-4
Compaction: Standard Proctor
FIG 44
VOID RATIO (e)

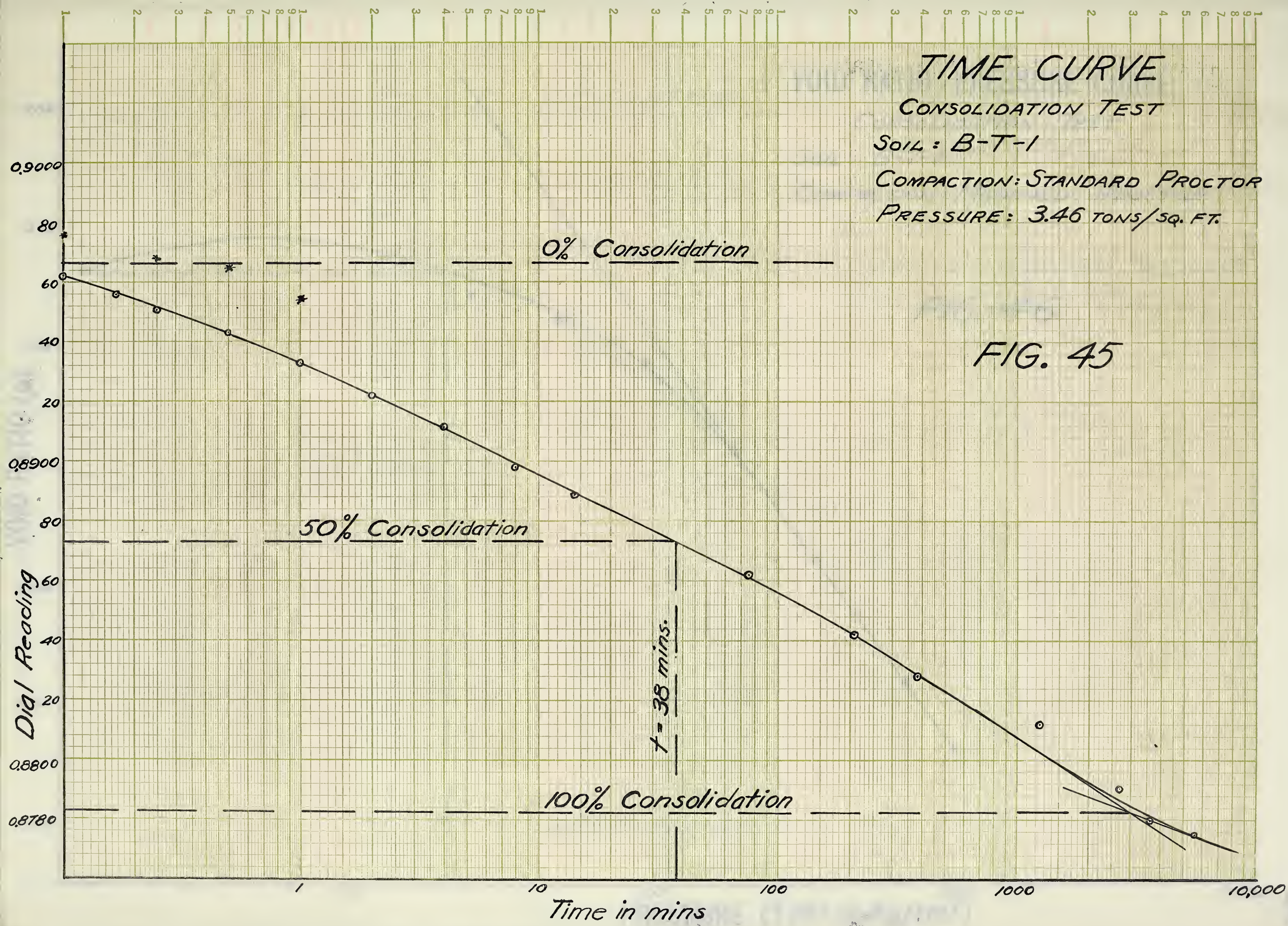

TIME CURVE
CONSOLIDATION TEST
SOIL : B-T-1
COMPACTION: STANDARD PROCTOR
PRESSURE: 3.46 TONS/SQ. FT.
0% Consolidation
50% Consolidation
100% Consolidation
t = 38 mins.
Dial Reading
0.9000
80
60
40
20
0.8900
0.8800
0.8780
Time in mins
1
10
100
1000
10,000

FIG. 45

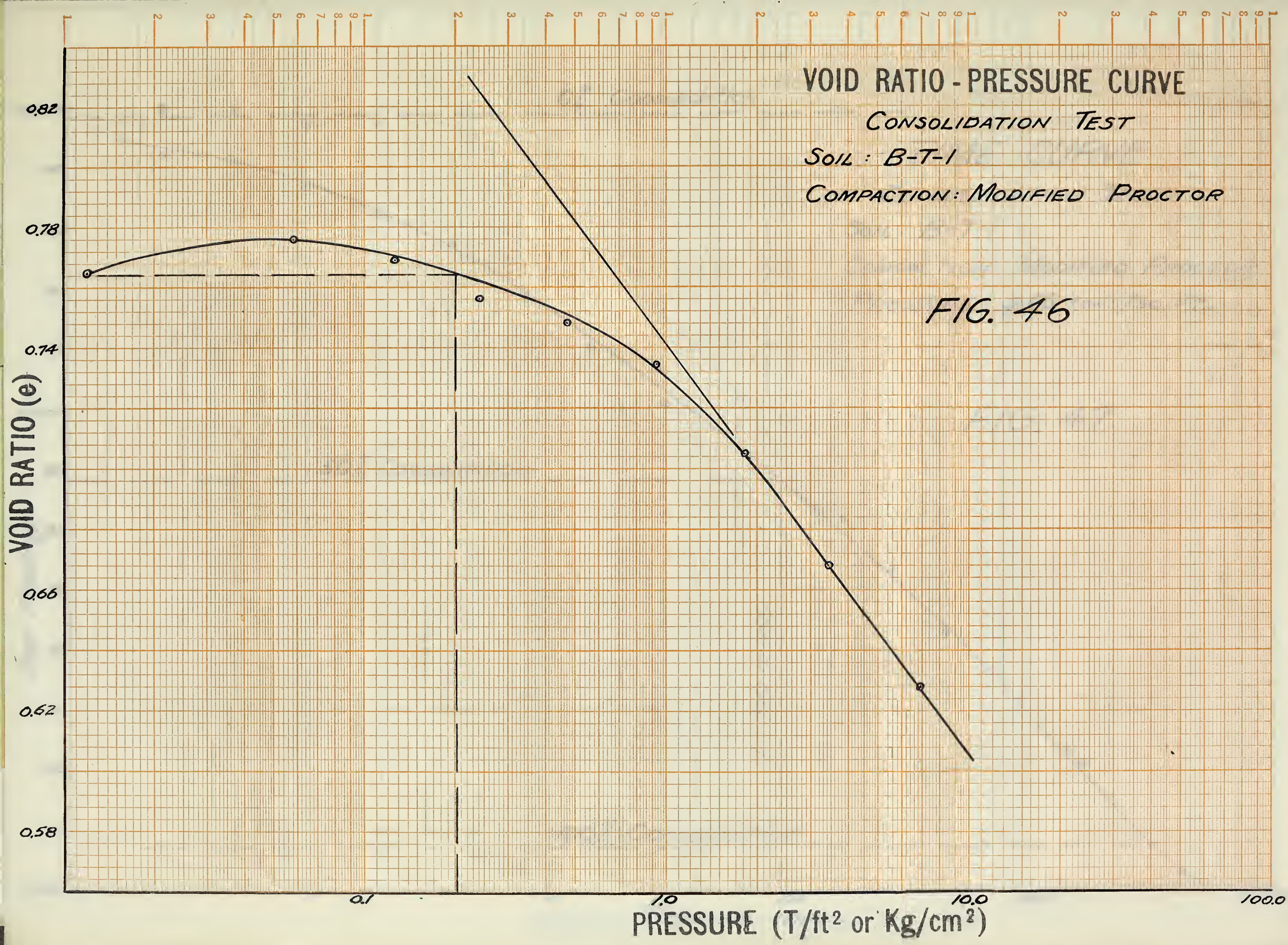
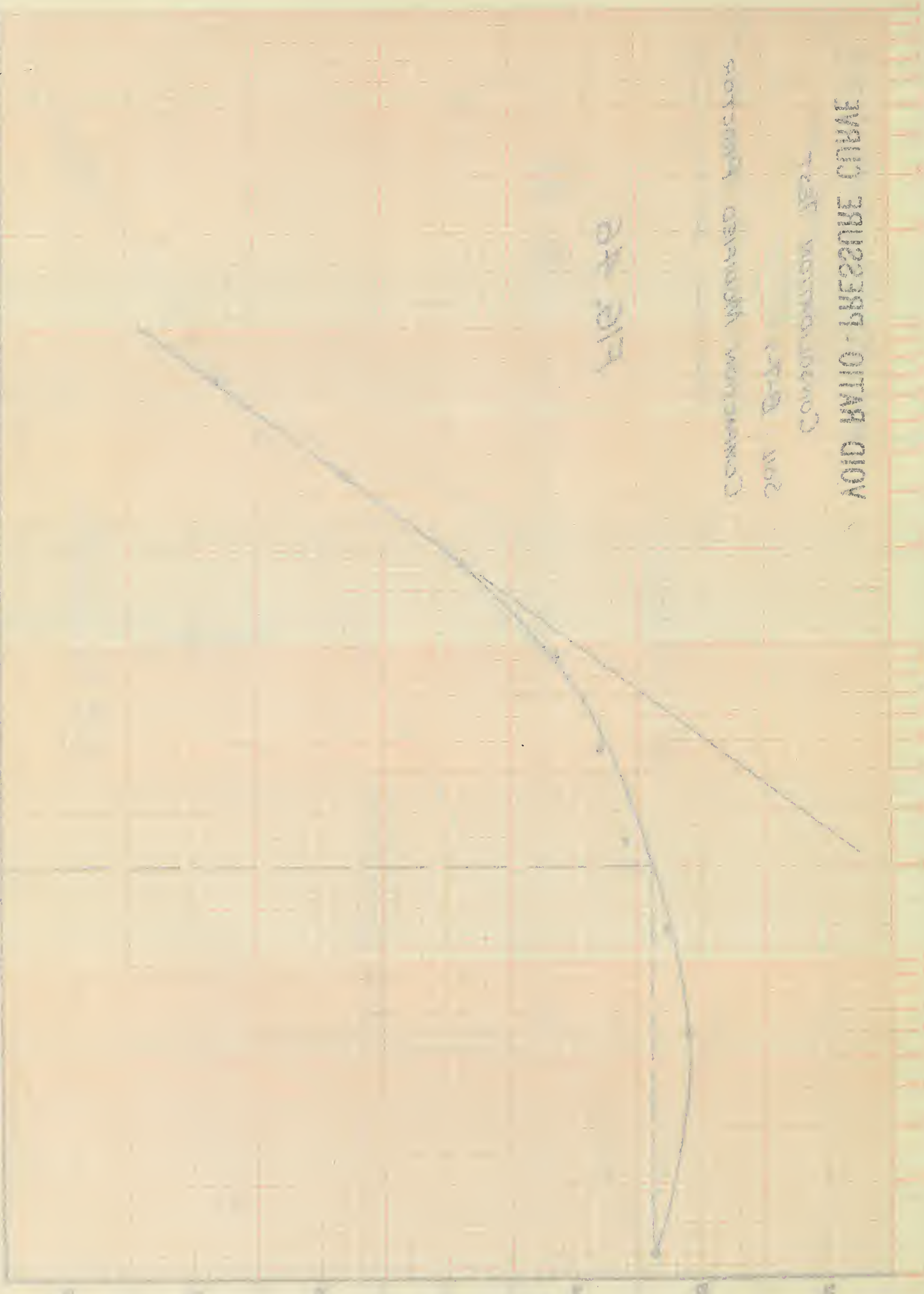

VOID RATIO - PRESSURE CURVE
CONSOLIDATION TEST
SOIL : B-T-1
COMPACTION : MODIFIED PROCTOR
VOID RATIO (e)
0.82
0.78
0.74
0.66
0.62
0.58
PRESSURE (T/ft² or Kg/cm²)
0.1
1.0
10.0
100.0

FIG. 46



VOID RATIO - PRESSURE CURVE
CONSOLIDATION TEST
Correction Modified Proctor
FIG #6
VOID RATIO (e)

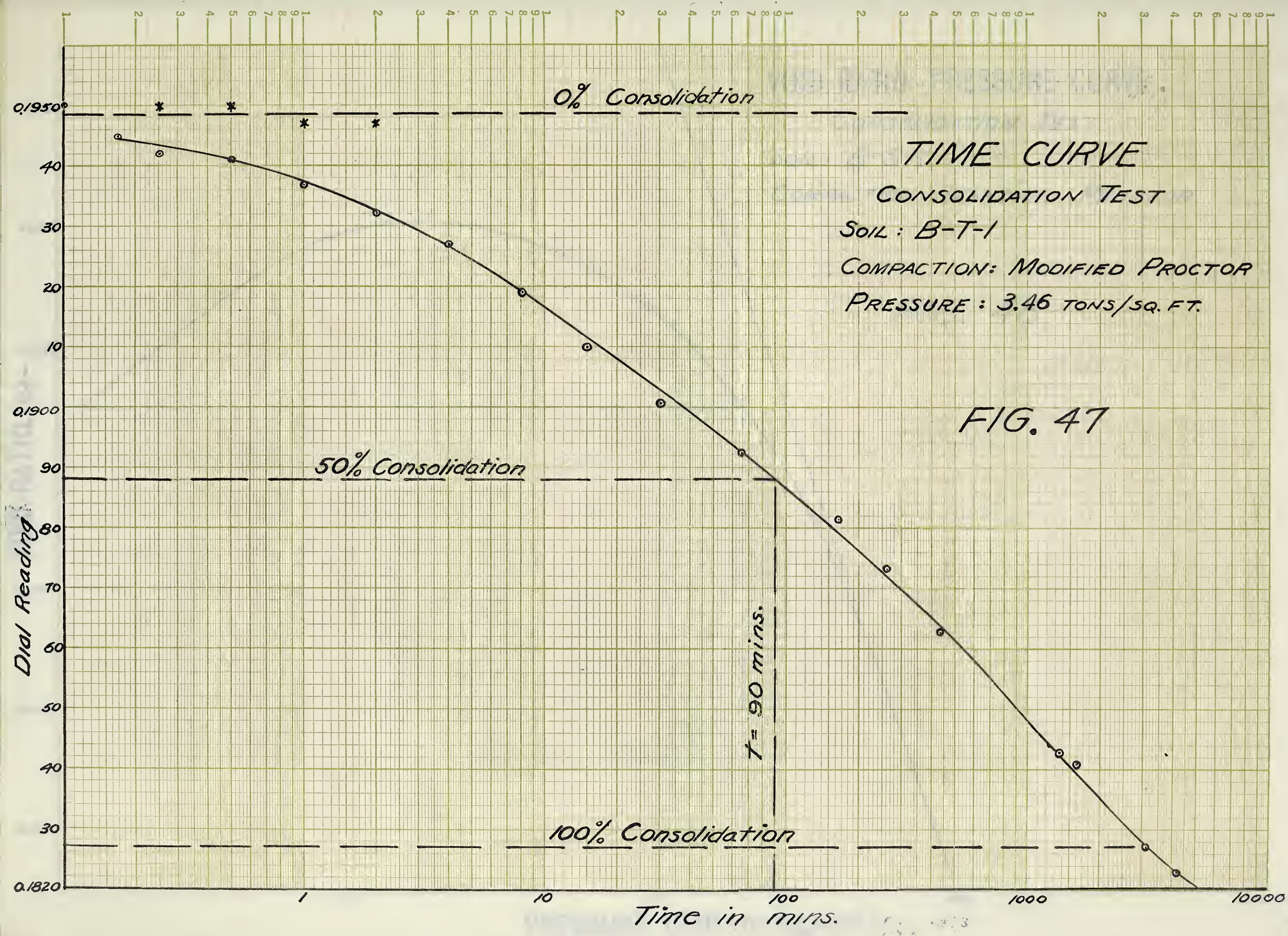
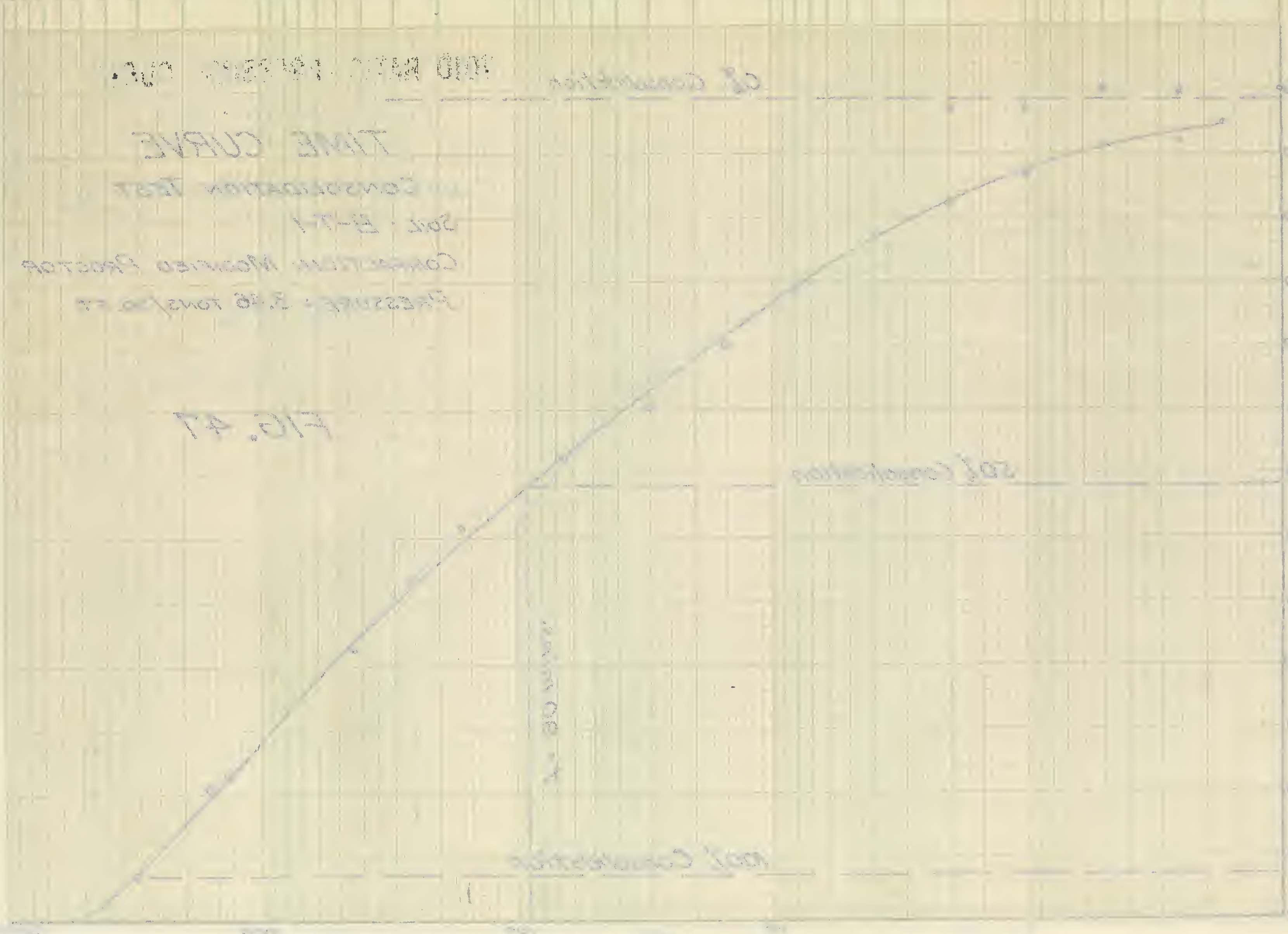
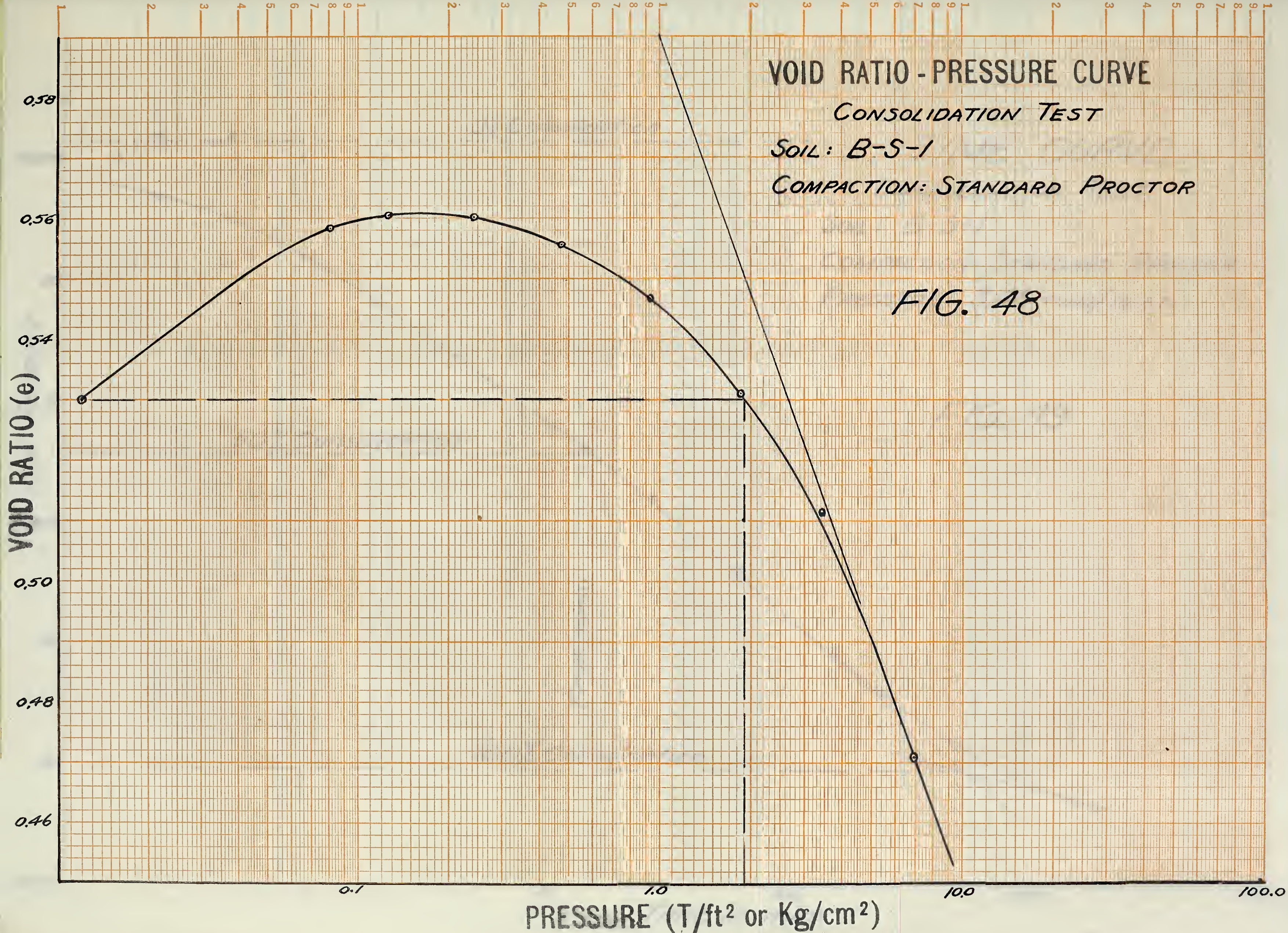

TIME CURVE
CONSOLIDATION TEST
SOIL : B-T-1
COMPACTION: MODIFIED PROCTOR
PRESSURE : 3.46 TONS/SQ. FT.
0% Consolidation
50% Consolidation
100% Consolidation
T = 90 mins.
Dial Reading
0.1950
40
30
20
10
0.1900
90
80
70
60
50
40
30
0.1820
Time in mins.
1
10
100
1000
10000

FIG. 47

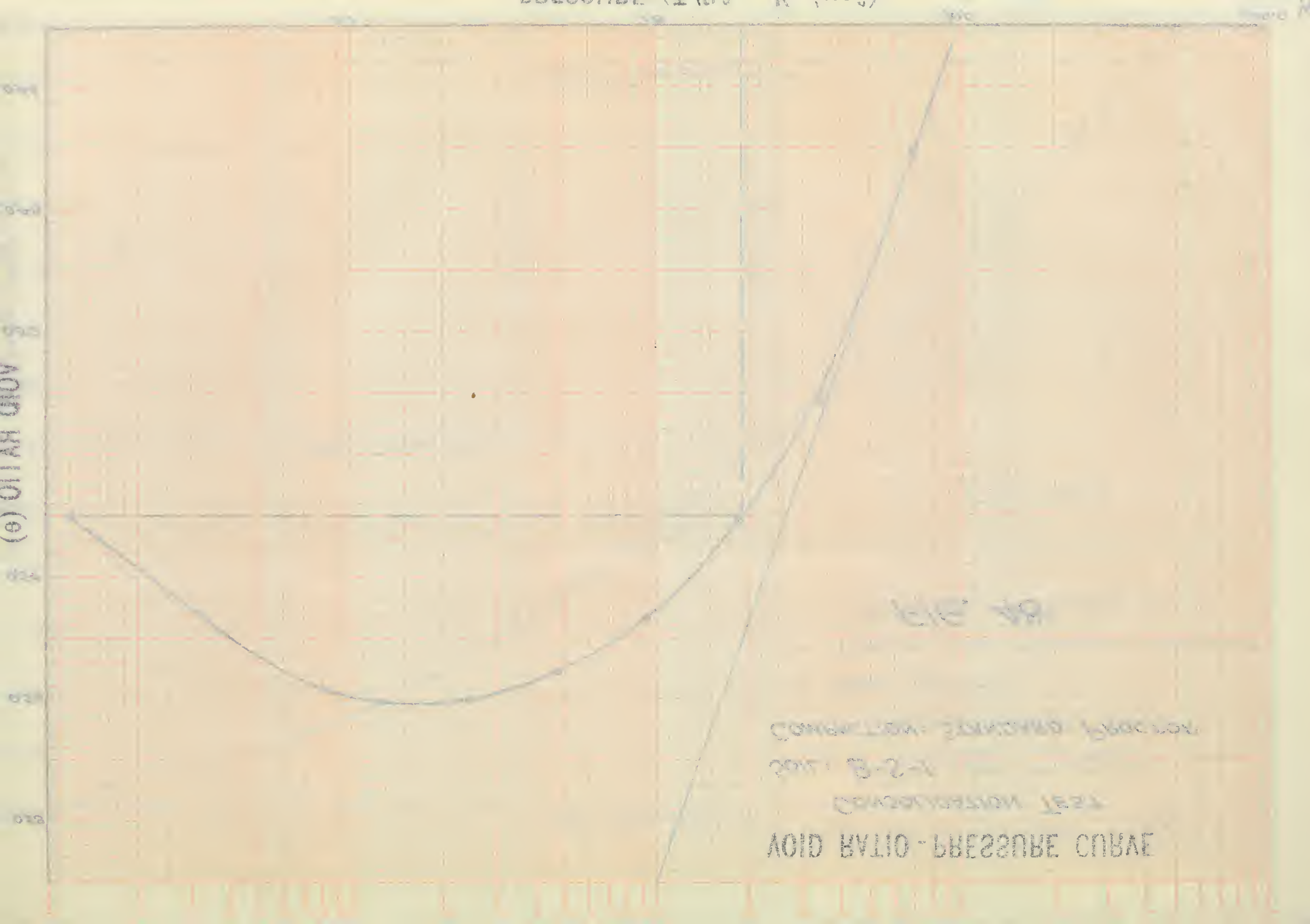


TIME CURVE
Consolidation Test
FIG. 47
0% Consolidation
50% Consolidation
100% Consolidation



VOID RATIO - PRESSURE CURVE
CONSOLIDATION TEST
SOIL: B-S-1
COMPACTION: STANDARD PROCTOR
0.58
0.56
0.54
0.50
0.48
0.46
VOID RATIO (e)
0.1
1.0
10.0
100.0
PRESSURE (T/ft² or Kg/cm²)

FIG. 48


VOID RATIO - PRESSURE CURVE
CONSOLIDATION TEST
VOID RATIO (e)

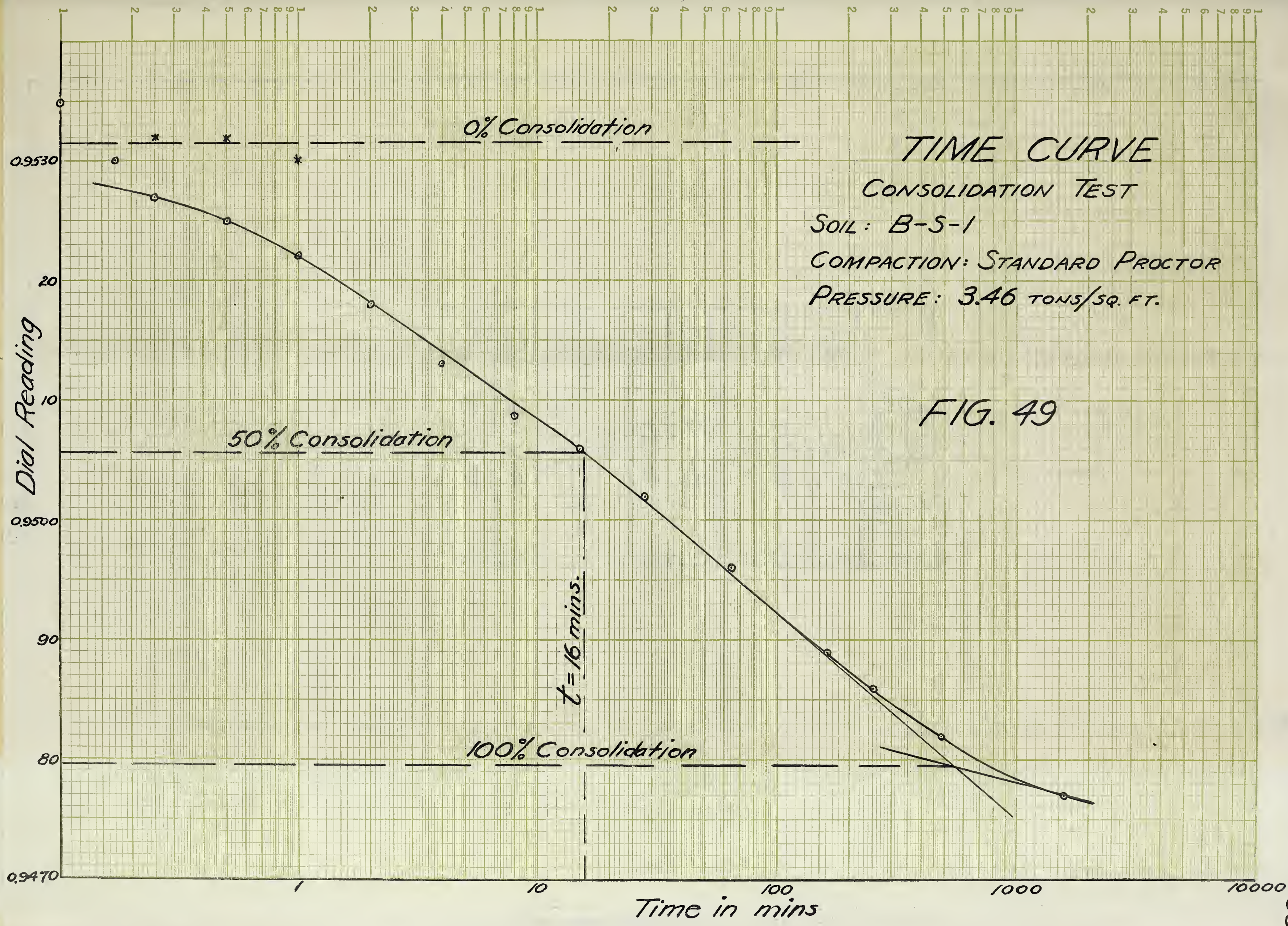
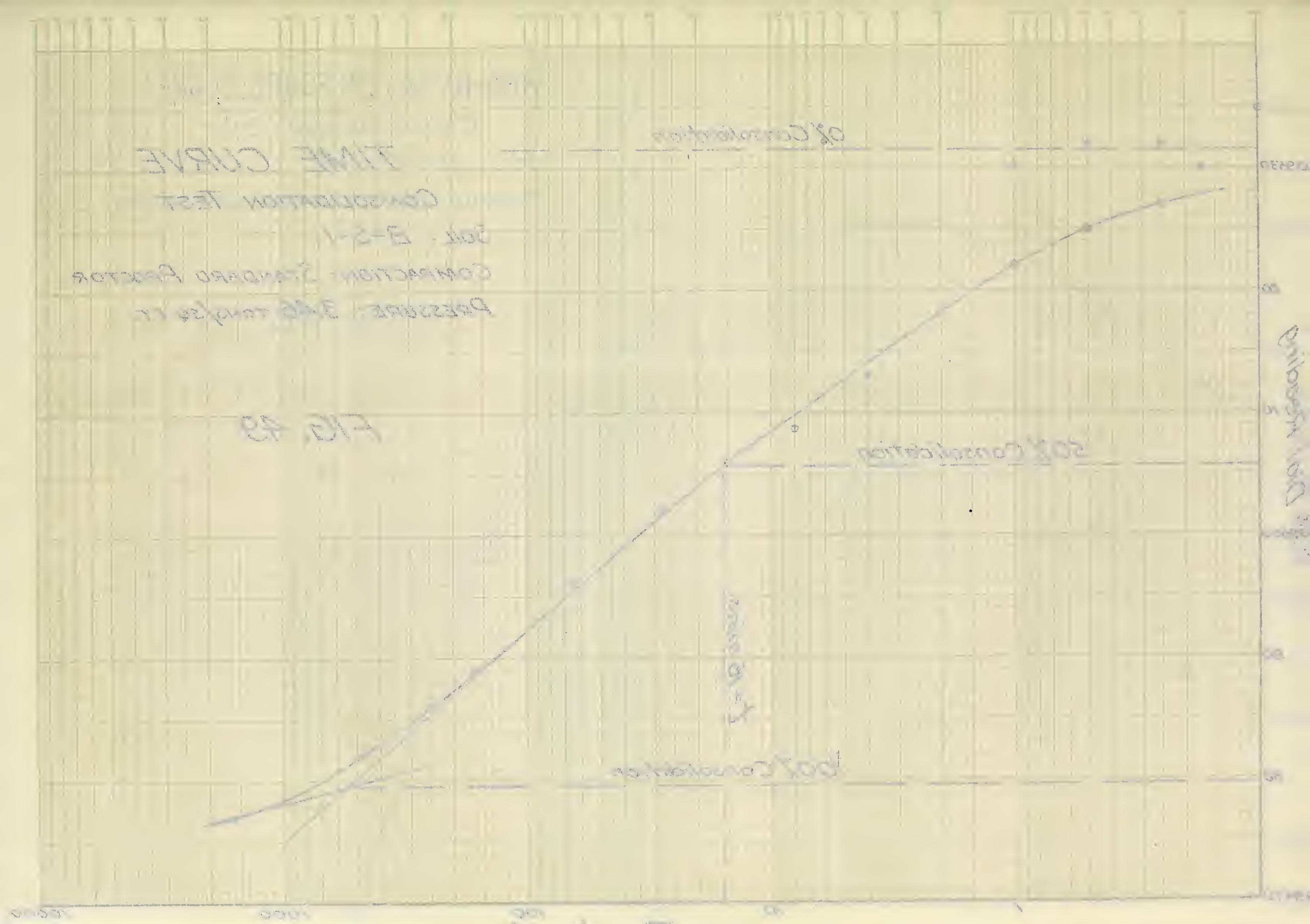

0% Consolidation
50% Consolidation
100% Consolidation
$t = 16$ mins.
Dial Reading
0.9530
20
10
0.9500
90
80
0.9470
Time in mins
1
10
100
1000
10000
TIME CURVE
CONSOLIDATION TEST
SOIL: B-S-1
COMPACTION: STANDARD PROCTOR
PRESSURE: 3.46 TONS/SQ. FT.

FIG. 49


TIME CURVE
Consolidation Test
Soil B-5-1
Compaction: Standard Proctor
Pressure: 3/16 tons/sq ft
FIG. 49
0% Consolidation
50% Consolidation
100% Consolidation
Dial Reading

CONSOLIDATION TEST RESULTS

	B-T-1		B-S-1
	STANDARD	MODIFIED	STANDARD
Moisture content for compaction	31.6	25.6	16.5
Moisture content (start of test)	32.9	27.2	16.8
Moisture content (end of test)	27.2	26.2	17.5
Dry density	79.5	87.0	107.8
Compressive index	0.20	0.13	0.14
Swelling pressure - Tons/sq. ft	0.14	0.21	1.91
Coefficient of permeability - cm/min	3.5×10^{-7}	2.7×10^{-8}	1.0×10^{-7}
Coefficient of compressibility - cm²/kg.	0.036	0.023	0.014
Coefficient of consolidation - cm²/min.	0.0046	0.0020	0.011

FIG. 50

IV

DISCUSSION

1. SPECIFIC GRAVITY TEST RESULTS

The specific gravity gives an indication of the type of soil. The results of the soils tested are listed in Figure 14. The subsoils have values ranging from 2.67 to 2.69, indicating sandy clays and silt-clays. The effect of organic material in the topsoils is quite apparent. The specific gravities of the topsoils range from 2.5 to 2.6, with the specific gravity of one topsoil, B-T-1, being 2.4. When warm this particular topsoil also emits the unmistakeable odor of decaying organic material. Thus, the author believes that the specific gravities of topsoils give an indication of the relative amount of organic material present.

2. GRAIN SIZE CURVES

Grain size analyses were made on the soils, A-T-1, B-T-1, and B-S-1. These results are shown in Figures 17 to 19. A. Casagrande states that the critical diameter of particles so far as frost heave danger is concerned is 0.02 mm. If the amount of these particles is less than 1 per cent, no heave is to be expected, while considerable heaves may take place if this amount is over 3 and over 10 per cent, in the case of nonuniform and very uniform soils, respectively. The following table, Figure 51, shows the per cent finer than 0.02 mm. for the three soils.

A-T-1	28%
B-T-1	41%
B-S-1	56%

Figure 51

The topsoil, A-T-1, is a nonuniform material, predominantly a silt-sand. The percentage finer than 0.02 mm. indicates that the soil is very susceptible to frost. However, the compacted material must possess a fair permeability if frost heave is to occur.

The topsoil B-T-1 is a fairly uniform soil, predominantly a clay-silt. The percentage finer than 0.02 mm. indicates that this soil might also be very susceptible to frost action. However, permeability computations from the consolidation tests for the material compacted at both standard and modified Proctor indicate that the rate of flow of water would be too slow to permit ice segregation.

The subsoil B-S-1 is a uniform clay-silt, with 56 per cent finer than 0.02 mm. The coefficient of permeability as computed from the consolidation test for standard Proctor density is fairly low, thus eliminating the possibility of frost heave.

3. ATTERBERG LIMIT TEST RESULTS

The results of the Atterberg limit test are tabulated in Figure 14. The liquid limit and plasticity index of each soil are plotted on the Plasticity Chart, Figure 15.

In general the soils tested plot above the A-line in the C I group of soils. The soils A-T-1, A-S-1, C-T-1, and B-T-1 do not plot above the A-line, thus indicating sandy silts or the presence of organic colloids.

The C I group of soils are inorganic clays of medium plasticity and compressibility, with medium to high dry strength. These soils are practically impervious.

The soils A-T-1 and A-S-1 plot in the soil groups ML-OL, being sandy silts, with slight to medium dry strength, and low compressibility.

The topsoil C-T-1 plots just below the A-line, due probably to the presence of organic material. Its physical characteristics would compare to those of the soils in the C I group.

The topsoil B-T-1 also plots below the A-line in the groups

MH-OH. The soil B-T-1 is thus an organic silt clay, with high compressibility. Its dry strength is fairly low and it is practically impervious to the flow of water. The distinctive odor of decomposing organic material is very strong when a wet sample of this topsoil is heated.

The compressibility is approximately proportional to the liquid limit. With soils having the same liquid limit, the toughness and dry strength increase and the permeability and compressibility decrease as the plasticity index increases. Thus the soil with the higher plasticity index is usually more suitable for fill construction.

A. Casagrande makes the following statement: "An important property of all fine-grained organic soils is the radical drop in plasticity due to oven-drying. This is due to the fact that organic colloids are very sensitive and undergo irreversible changes upon drying. Oven-drying also affects the limits of inorganic soils, but to a much more limited extent." He cites an example of an organic clay where the liquid limit dropped from 84 to 51 and the plasticity index from 34 to 9 upon oven-drying of the soil. Thus it is not advisable to dry soil samples before performing the limit tests.

The shrinkage limits of the soils tested range from 12 to 20, with one exception; the topsoil B-T-1 having a shrinkage limit of 31.5 per cent. This high value of the shrinkage limit of B-T-1 is quite significant, since it is not likely that the field moisture content of the embankment will ever be much higher than 30 per cent. Thus this topsoil would not exhibit any appreciable swelling or shrinkage. The remaining soils have much lower shrinkage limits and very possibly would exhibit appreciable swelling and shrinkage.

The position of a soil on the Plasticity Chart provides considerable information about its physical properties when in place.

This information has been summarized in Figure 16.

The author believes that if a topsoil and subsoil from the same place are in the same general soil group, they have quite similar physical properties. This statement is partly borne out by the fact that the topsoil and subsoil from location A have comparable optimum moisture contents, maximum densities and shearing strengths. These properties are quite different for the topsoil and subsoil from location B. A-T-1 and A-S-1 belong to the soil groups, ML-OL. The topsoil B-T-1 belongs to the groups, MH-OH. The subsoil B-S-1 belongs to an entirely separate group, C I.

4. PROCTOR DENSITY TEST RESULTS

The Proctor density test curves are shown in Figures 20 to 23, and the results of these tests are tabulated in Figure 24.

The topsoil and subsoil of location A have comparable compaction properties, the optimum moisture content and maximum dry density being similar for the two soils. Also significant is the fact that the above properties of the two soils are almost identical with modified Proctor compaction.

As expected from the classification test results, the compaction characteristics of the topsoil, B-T-1, are quite different from those of the subsoil, B-S-1. The organic material in the topsoil resists the compactive effort, explaining the resulting low maximum density. Because of these great differences, it was decided to compare other properties of the two soils. The stress and deformation properties of the two soils from the location A were also determined because of their similar compaction properties.

The topsoil F-T-1 belongs to the C I group of soils. Therefore, one might expect a modified Proctor dry density of 110 to 115 lbs. per cu. ft., similar to the two soils, B-S-1 and C-S-1. However, this

dry density is only 97 lbs. per cu. ft. In performing the classification tests, only the material passing a No. 40 sieve was used. This soil appears to have been taken from a cultivated field, as visual observation disclosed the presence of large amounts of straw. This probably accounted for the relatively low density, as material used in the Proctor test had to pass only a $\frac{1}{4}$" sieve.

5. TRIAXIAL COMPRESSION TEST RESULTS

During construction of an embankment the water content of the clay remains practically unchanged. Therefore, the shearing resistance of such clays immediately after construction corresponds to that determined from quick-shear tests, made on samples at the water content that the clay will have in the finished embankment immediately after construction. If the material contains air, as is usually the case in highway embankments, the shearing resistance increases with increasing normal stress.

Therefore, in this laboratory study, the shear characteristics of the soils from locations A and B have been determined by quick-shear tests.

The stress-strain curves and Mohr circles are plotted in Figures 25 to 33, the results being tabulated in Figure 34.

From this data, it is apparent that modified Proctor compaction as compared to standard Proctor compaction results in an increase of from 65 to 220 per cent in the cohesion and an increase of from 30 to 210 per cent in the angle of internal friction. In highway embankments, where the fill is usually not over 10 feet, the resulting confining pressure is approximately 0.5 tons per sq. ft. as a maximum.

In the following table, Figure 52, the shearing strengths have been evaluated by use of the following formulae obtained from the

geometry of the Mohr circle,

$$q_c = p_c (N_\phi - 1) + 2c\sqrt{N_\phi} \quad \ldots\ldots\ldots \quad (1)$$

and

$$s_c = 0.5\ q_c \cos \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

When the confining pressure $p_c = 0$,

$$q_u = 2c\sqrt{N_\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

and

$$s_u = 0.5\ q_u \cos \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

The above symbols are defined as follows:

q_c = confined compressive strength

= diameter of Mohr circle, $\sigma_1 - \sigma_3$

p_c = confining or lateral pressure

s_c = confined shearing strength

N_ϕ = flow value = $\tan^2 (45 + \frac{\phi}{2})$

c = cohesion = intercept on the vertical stress axis

ϕ = angle of internal friction

q_u = unconfined compressive strength

s_u = unconfined shearing strength

It must be remembered that the values of q and s tabulated in Figure 52 are the ultimate compressive and shearing strengths, respectively.

In other words, no attempt has been made to take into account the compressibility of the material in place. Repetitive field bearing tests, approximately actual highway loading, should permit an estimate of the detrimental deflections in the subgrade. It is the opinion of the author that the results of these bearing tests could then be correlated with the moduli of deformation or other quantitative results of traxial compression tests.

SHEARING STRENGTH EVALUATION

Soil Number	Proctor Compaction	Depth of Soil in Fill: 0 ft, $p_c = 0$		5 ft, $p_c = 0.20$ (approx)		10 ft, $p_c = 0.40$ (approx)	
		q_u	s_u	q_c	s_c	q_c	s_c
A-T-1	Standard	1.74	0.81	1.99	0.92	2.24	1.04
	Modified	3.25	1.42	3.64	1.59	4.03	1.75
A-S-1	Standard	1.31	0.61	1.57	0.72	1.84	0.85
	Modified	5.45	2.21	6.01	2.44	6.58	2.67
B-T-1	Standard	2.19	1.01	2.45	1.13	2.71	1.25
	Modified	4.97	2.15	5.37	2.32	5.78	2.50
B-S-1	Standard	3.78	1.86	3.88	1.91	3.97	1.95
	Modified	9.50	3.91	10.03	4.15	10.55	4.35

Figure 52

Note: All units are tons per sq. ft.

From a study of the results of Figure 52, the following conclusions are apparent:

(a) The shearing strength of the sandy silt topsoil A-T-1, compacted at standard Proctor, is approximately 125 per cent of the shearing strength of the sandy silt subsoil A-S-1, compacted at standard Proctor.

(b) The shearing strength of A-T-1, compacted at modified Proctor, is approximately 65 per cent of the shearing strength of A-S-1, compacted at modified Proctor.

(c) The shearing strength of A-T-1, compacted at modified Proctor, is approximately 225 per cent of the shearing strength of A-S-1, compacted at standard Proctor.

(d) The shearing strength of the highly organic clay topsoil B-T-1, compacted at standard Proctor is approximately 60 per cent of the shearing strength of the silt-clay subsoil B-S-1, compacted at standard Proctor.

(e) The shearing strength of B-T-1, compacted at modified Proctor, is approximately 55 per cent of the shearing strength of B-S-1, compacted at modified Proctor.

(f) The shearing strength of B-T-1, compacted at modified Proctor, is approximately 120 per cent of the shearing strength of B-S-1, compacted at standard Proctor.

(g) An increase of the compactive effort from standard to modified Proctor, results in the following percentage increases in shearing strength, as listed in Figure 53.

Soil Number	Type of Soil	Percentage Increase
A-T-1	sandy silt topsoil	70
A-S-1	sandy silt subsoil	240
B-T-1	highly organic clay topsoil	105
B-S-1	silt clay subsoil	115

Figure 53

From the data of Figure 34 and the stress-strain curves, Figures 25 to 33, the moduli of deformation and degree of saturation have been estimated and tabulated below in Figure 54.

Soil Number	Standard Proctor		Modified Proctor	
	Mod. of Def.	Deg. of Sat.	Mod. of Def.	Deg. of Sat.
A-T-1	370	0.65	370	0.57
A-S-1	280	0.91	450	0.77
B-T-1	150	0.85	150	0.85
B-S-1	170	0.85	600	0.67

Figure 54

From these results it is apparent that an increase in the compactive effort from standard to modified Proctor results in:

(a) No change in the modulus of deformation for the topsoils.

(b) An increase of approximately 60 per cent in the modulus of

deformation of the sandy silt, A-S-1.

(c) An increase of approximately 250 per cent in the modulus of deformation of the silt-clay, B-S-1.

(d) A small decrease in the degree of saturation of the sandy silt topsoil, A-T-1.

(e) No change in the degree of saturation of the organic clay topsoil, B-T-1.

(f) A considerable decrease in the degree of saturation of the subsoils.

It must be remembered that the modulus of deformation permits only an estimate of the deformation due to shearing stresses in the material. An attempt is made to evaluate the deformation due to compressibility in the discussion of the results of the consolidation tests.

In the Department of Transport publication, "Airport Runway Evaluation in Canada", by Dr. N. McLeod, design curves for highway wheel loadings are presented. In the following table, Figure 55, the required thicknesses of granular base are listed as determined using the values of the angles of internal friction of Figure 34, and a wheel load of 7,500 lbs.

Soil Number	Standard Proctor	Modified Proctor
A-T-1	7	3
A-S-1	7	outside range of curves 0 thickness
B-T-1	7	4
B-S-1	11	outside range of curves 0 thickness

Figure 55

Note: A minimum thickness of 6 inches is recommended.

Correlation of the results of Figure 55 to the shearing strengths and moduli of deformation of Figures 52 and 54 is just not possible. Since these design curves were determined by the use of plate bearing tests, one would expect correlation to the strength and deformation properties of the soil.

6. CALIFORNIA BEARING RATIO TEST RESULTS

The C.B.R. curves are shown in Figures 37 to 39. Results of the C.B.R. tests have been tabulated in Figure 40.

The C.B.R. test is a penetration shear test used to determine a modulus of the shearing resistance of soils. Thus it should not be used where factors other than the shearing resistance control the design.

The U. S. Waterways Experiment Station recently concluded an investigation of the C.B.R. method of design of flexible pavements. The results are published in the Manual, "The California Bearing Ratio Test as Applied to the Design of Flexible Pavements for Airports". From the conclusions of this report, the author believes that, at the present time, C.B.R. tests on remolded samples would not provide an answer to the question of whether or not topsoils can be used in highway embankments for the following reasons:

(a) Wide variations in C.B.R. test results on compacted material occur. These variations are caused primarily by the effects of water content, density and type of compaction used in preparing samples; small changes in density and molding water content greatly affecting the C.B.R. value.

(b) Since the molding water content greatly affects the physical properties of soils, it follows that, in remolded soils, duplicate laboratory samples cannot be prepared, unless the same molding water

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

content and method of compaction are duplicated, even though water content and densities obtained subsequent to molding are duplicated.

(c) C.B.R. tests on natural undisturbed samples and on remolded samples with the same molding water content, moisture conditions, density and physical properties that will occur during and after construction are recommended. However, the laboratory study indicated a marked difference in the physical properties of a soil prepared by different compaction methods in the laboratory, and it can be assumed that these differences occur when the soil is compacted by different methods in the field.

(d) Stress-penetration curves of compacted laboratory samples are generally concave upward for the range from approximately 0 to 0.2 inches of penetration. The manual recommends correcting such stress-penetration curves. A curve of this type and the correction is shown in Figure 38. The resulting increase in the soaked C.B.R. value due to the correction is almost 90 per cent. It is the author's opinion that this correction is possibly not warranted. From the set of empirical design curves, for a wheel load of 7,500 lbs., published in the U. S. Waterways Experiment Station Manual, the resulting decrease in thickness of base course and pavement is about 10 inches, which decrease represents a considerable saving in construction costs.

From the data of Figure 40 the following significant conclusions are apparent:

(a) The organic clay topsoil, when compacted at standard Proctor, actually consolidates under the weight of the soaking surcharge. This surcharge approximates a 6-inch thickness of base course and surface.

(b) The topsoil compacted at modified Proctor and the subsoil compacted at standard Proctor exhibit the same swelling under the weight of the soaking surcharge.

(c) From the U. S. Waterways Experiment Station's empirical design

curves for a sheel load of 7,500 lbs., the following combined thicknesses of base course and pavement are required, Figure 56.

Soil Number	Standard Proctor	Modified Proctor
B-T-1	outside range of design curves	17
B-S-1	8	--

Figure 56

(d) From the Department of Transport's design curves for a wheel load of 7,500 lbs., the following thicknesses of base course are required, Figure 57:

Soil Number	Proctor (Standard)	Proctor (Modified)
B-T-1	outside range of design curves approx. 23	16
B-S-1	outside range of design curves 0 thickness	--

Note: A minimum thickness of 6 inches is recommended.

Figure 57

The results of Figures 56 and 57 definitely indicate that the organic clay topsoil cannot be used directly under the base course. There would seem to be no objection to placing the topsoil in the bottom of the fill, provided the compressibility is not excessive.

7. FREEZING TEST RESULTS

The results of the freezing tests on the highly organic topsoil, B-T-1 and the silt-clay, B-S-1, are tabulated in Figure 41.

The following conclusions are drawn from these results:

(a) The expansion upon freezing cannot be correlated to the moisture content, the density, or the void ratio of the material in place.

(b) As might be expected, this expansion does bear a relation to the degree of saturation of the material, since the degree of saturation is the ratio of the volume of water to the volume of voids. When this ratio is low, there is no expansion of the soil mass upon freezing. Instead, the expansion of the water upon freezing takes place in the large air space. When this ratio is high, all of the expansion cannot take place in the available air space and an increase in the volume of the soil mass must result.

(c) Soaking the sample results in an increase of the degree of saturation and accordingly, an increase in the expansion of the soil mass upon freezing.

(d) The higher compactive effort, modified as compared to standard Proctor, results in an increase of the expansion of the soil mass upon freezing. This is also expected, since the higher compactive effort is effective in reducing the volume of air voids, available for expansion to occur in.

In general, it would seem that the topsoil is not as desirable a material as the subsoil, from the standpoint of the effects of freezing.

8. MOISTURE CONTENT-COMPRESSIVE STRENGTH RELATIONSHIPS

The U. S. Waterways Experiment Station in their publication, "Triaxial Shear Research and Pressure Distribution Studies on Soils", has succeeded in obtaining a fairly consistent relation between final water content and compressive strength of saturated undisturbed clays. This relation shows the characteristic shape of a pressure-void ratio curve,

and is approximately parallel to the consolidation test curve. In the investigation of compacted soils, the Waterways Experiment Station arrived at the following conclusions:

(a) The compressive strengths for tests under different lateral pressures, when plotted against water content at end of test, do not fall on, or close to, a single curve. The test points fall on curves or in groups that are displaced for each different lateral pressure.

(b) The compressive strengths from quick and quick-consolidated tests under comparable conditions are not affected by the type of test. In other words the average results of quick and quick-consolidated tests are the same.

In this laboratory study, the author performed unconfined compression tests on soaked samples of the topsoil, B-T-1, compacted at optimum standard and modified Proctor. The compression test specimens were allowed to dry to varying degrees of moisture content, thus resulting in varying values of the compressive strength. The results are shown in Figures 42 and 43. A study of these curves discloses the following:

(a) The moisture content compressive strength curves are not parallel to the consolidation test curves.

(b) A break in the moisture content-compressive strength curve occurs at approximately 28 per cent moisture content. The shrinkage limit of the topsoil, B-T-1, is 31.5 per cent. It is quite possible that the shrinkage limit had decreased to 28 per cent, when the compression tests were performed, because of the progressive decomposition of the organic matter in the topsoil. This break in the curve at the shrinkage limit can be explained by theory if the effect of progressive drying from above the shrinkage limit is considered. A. Warlam obtained similar results in his thesis for a Ph.D. degree from Harvard University.

(c) The compressive strength of the topsoil compacted at modified

Proctor for a moisture content of 26.6 per cent checks with the unconfined compressive strength result of Figure 34. This value of 26.6 per cent is below the shrinkage limit.

(d) The compressive strength of the topsoil compacted at standard Proctor does not check with the unconfined compressive strength result of Figure 34. However, if the portion of the curve beyond the shrinkage limit is projected back, the strength at 32.5 per cent moisture then does check with the result of Figure 34.

This would seem to indicate that increasing degrees of saturation result in a rapidly increasing rate of decrease of the shearing strength.

It would also indicate that if the degree of saturation or moisture content does not increase, the portion of the curve to the left of the shrinkage limit is not correct. Instead a straight line relationship on semi-logarithmic paper exists.

(e) An increase in the compactive effort from standard to modified Proctor results in an increase of the strength. As the material dries the percentage increase in shearing strength, because of the higher compactive effort, gradually increases.

9. CONSOLIDATION TEST RESULTS

The pressure-void ratio and time-consolidation curves are plotted in Figures 44 to 49, the results being tabulated in Figure 50.

A study of these results discloses the following:

(a) The compressive index of the topsoil, B-T-1, compacted at standard Proctor is approximately 50 per cent higher than the compressive index when compacted at modified Proctor.

(b) This compressive index of the topsoil compacted at modified Proctor is approximately equal to the compressive index of the subsoil compacted at standard Proctor.

(c) The swelling pressure of the topsoil compacted at standard

Proctor is approximately equal to the swelling pressure when compacted at modified Proctor, both values being very low.

(d) The swelling pressure of the subsoil compacted at standard Proctor is quite high, being approximately 10 times the swelling pressures of the topsoil for both standard and modified Proctor compaction.

The weight of a fill increases the pressure on a confined stratum of soil from the overburden pressure p_o to the value $p_o + \Delta p$. The corresponding void ratio decreases from e_o to e. Then the compression S of the confined stratum is

$$* \quad S = H \frac{C_c}{1+e_o} \log_{10} \frac{p_o + \Delta p}{p_o}$$

where H = thickness of the bed of soil

C_c = compressive index

For the purpose of comparing the topsoil compacted at both standard and modified Proctor and the subsoil compacted at standard Proctor, let us assume identical loading conditions for each and equal values of H. The quantity

$$H \log_{10} \frac{p_o + \Delta p}{p_o}$$ is thus constant for the three cases.

Therefore, $S = K \dfrac{C_c}{1+e_o}$

Comparing the topsoil compacted at modified Proctor and the subsoil compacted at standard Proctor, the values of C_c are almost identical for both so that the compression S is thus inversely proportional to the void ratio. The topsoil possesses the greater void ratio, ?

* From--"Soil Mechanics in Engineering Practice"--Terzaghi and Peck.

and accordingly the compression of this soil will be appreciably less than that of the subsoil.

Comparing the topsoil and subsoil compacted at standard Proctor, the compressive index of the topsoil is almost 50 per cent higher than the compressive index of the subsoil, while the void ratio of the topsoil is almost 50 per cent higher than that of the subsoil for all pressures within the straight line portions of the pressure-void ratio curves. The net result is that the compression of the both soils will be almost equal.

V

CONCLUSIONS

1. In general, this laboratory study would indicate that organic topsoils may be placed in an embankment, provided they are compacted to a density approximating maximum modified Proctor density. In order to satisfy the design requirements of the C.B.R. test, the material should be placed in the bottom of the fill. This procedure is very practical, since the topsoil is the first soil to be handled in building the grade.

2. The physical properties of the topsoil, when compacted at a density slightly lower than that of maximum modified Proctor density, would have values somewhere between the values obtained for maximum standard and maximum modified Proctor density. The results of this laboratory study would indicate that such a deviation from maximum density is permissible. However, this deviation should be held to a minimum.

3. The effects of decomposition of the organic matter have not been studied. It is quite possible that the compressibility of a highly organic topsoil in an embankment may increase as decomposition of the organic matter occurs. Also, at the present time, it is not known how closely the physical properties of laboratory samples correspond to those of the material in place.

4. Incidental to the main problem, the following conclusions are also drawn:

   (a) The specific gravity of a topsoil gives an indication of the amount of organic material present.

   (b) The position of a topsoil or subsoil on the Plasticity Chart

provides considerable information as to its physical properties.

(c) Highly organic topsoils have very low densities and fairly high optimum moisture contents. The organic material resists the compactive effort.

(d) An increase in compactive effort from standard to modified Proctor results in an increase of both the cohesion and angle of internal friction, these increases varying with the type of soil. The cohesion is the dominant factor in the shearing strength of highway embankments.

(e) An increase in the compactive effort from standard to modified Proctor results in an approximate increase of 100 per cent in the shearing strength.

(f) An increase in the compactive effort from standard to modified Proctor results in no appreciable change of the shearing deformation characteristics of a topsoil. A considerable improvement results in the case of a silt or clay subsoil.

(g) The C.B.R. test definitely indicates the impossibility of economically using a highly organic topsoil directly under the base course and pavement. At the present time, it would seem that there are too many variables associated with the test to permit its use for the purpose of design.

(h) A relationship does exist between the moisture content at the end of the test and the shearing strength of compacted soils. A definite break in this relation occurs at a moisture content approximately equal to the shrinkage limit. As a soil compacted at optimum moisture content dries, the shearing strength rapidly increases. There must be some point beyond which the shearing strength does not increase any more.

(i) The swelling pressures exerted by compacted highly organic topsoils are relatively low.

(j) An increase in the compactive effort from standard to modified Proctor is very effective in decreasing the compressibility of a highly organic topsoil.

VI

RECOMMENDATION FOR FURTHER STUDIES

Further investigation is necessary to determine the effects of time on highly organic topsoils. This should be possible by correlating field tests to a laboratory study.

It is the opinion of the author that this investigation should include:

1. A soil survey program along existing highways. This survey should be concentrated chiefly within apparent general failure zones.
2. Field density and moisture content determinations.
3. A laboratory study on:
   (a) Undisturbed samples of both the topsoil and subsoil from apparent failure zones.
   (b) Disturbed samples of both the topsoil and subsoil from locations corresponding to the undisturbed samples.
4. The construction of test sections and observation of the effects of traffic, time, and climatic conditions on these sections.

By means of classification and strength tests, correlation of the field and laboratory study should be possible. Once this correlation has been obtained, it will be necessary to perform classification and strength tests on only disturbed samples from the proposed right-of-way. It may be that classification tests alone will provide the answer to the question of using organic topsoils in highway embankments.

VII

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